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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

THE GREASE PROBLEM IN SEWAGE TREATMENT

By Almon L. Fales[1] and Samuel A. Greeley,[2] Members, Am. Soc. C. E.

Synopsis

Until recent years, municipalities in the United States, situated on the seacoast, have had relatively little difficulty in disposing of their sewage with scant, if any, treatment. In general, the most that was required of such municipalities was the construction of intercepting sewers (to prevent the discharge of sewage from numerous small outlets at the shore) and one or more outfall sewers extending to deep water for disposal by dilution. With increasing population and higher standard of cleanliness, it has become increasingly difficult to dispose of sewage in this manner without creating conditions objectionable to bathing, boating, fishing, and the like.

The offensive conditions have been due primarily to deposits of settling suspended matters and to floating sewage matters, including grease and oil which form "sleek" on the surface. The result in many cases has been that steps have been taken to provide sewage treatment plants for clarification and grease removal. In view of the important relation of grease to the problem of sewage treatment and disposal, the purpose of this paper is to discuss the problems of grease removal in sewage.

Definition of Grease

A standard concept of grease in sewage is that it "includes fats, waxes, free fatty acids, calcium and magnesium soaps, mineral oils, and other non-fat materials" (1).[3] When "grease" is mentioned in connection with sewage and its treatment, almost any combination of these substances may be meant. Sometimes, however, the liquid portions are called "oil" and the solid portions "grease." The term "grease" is generally applied to all these materials whether of animal or vegetable origin.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June, 1942. Opening discussion on this paper appears elsewhere in this issue.

[1] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[2] (Greeley & Hansen), Chicago, Ill.

[3] Numerals in parentheses, thus: (1), refer to corresponding items in the Bibliography (Appendix).

Sources of Grease in Sewage

In a broad sense, there are three sources of the grease and oil in sewage—domestic, industrial, and automotive. Domestic sources of grease include dwellings, hotels, restaurants, clubs, and institutions, the grease originating in human feces and kitchen slops. Feces contain fatty acids, soap, and neutral fat, whereas kitchen slops contain not only the unused fats from foodstuffs, such as lard, suet, butter and olive oil, but also soaps. The latter include both the sodium soaps and the calcium and magnesium soaps, or mineral soaps, which result from the reactions due to the hardness of the water supply.

Industrial wastes may contain large quantities of grease and oil, particularly those from abattoirs, packing houses, wool-processing plants, rayon plants, tanneries, oil refineries, gas works, canning plants, dairies, bakeries, and laundries.

Automotive sources of oil and grease include garages, repair shops, and automobile-washing plants. Although the larger garages may collect such material and prevent it from entering the sewerage system, many individual owners discharge waste of this kind into the sewers.

Characteristics of Grease

True fats are compounds of the alcohol glycerol (commonly called glycerin) with the fatty acids, such as oleic, palmitic and stearic, the resulting glycerids being designated as olein, palmitin and stearin (2). Other glycerids are also known. The three mentioned are found in animal fats—the first being a liquid and the others solids.

Fats contain carbon, hydrogen, and oxygen in varying proportions, the chemical structure of the molecules being relatively simple. Fats are among the more stable of organic compounds. The mineral acids attack them, with the result that glycerin and fatty acid are formed. In the presence of alkalis, such as sodium hydroxide, glycerin is liberated and alkali salts of the fatty acids are formed. The latter are known as soaps and, like the fats, are of stable character. Common soaps, soluble in water, are made in the manner indicated, by saponification of fats with sodium hydroxide. Mineral soaps, on the other hand, are insoluble and are precipitated. Soaps in sewage are largely of the latter kind.

The grease in sewage may be either "free" or "combined." The free grease is partly of mineral origin, such as gasoline, kerosene, and fuel oil, and partly of organic origin, such as lard, butter, and cottonseed oil. Most of the combined grease in sewage is soap. Although oil and water are immiscible, one can be dispersed in the other with the aid of a third substance, or emulsifying agent, to form an emulsion. In sewage, oil and water form emulsions, with the aid of soaps and, in this manner, much of the "free" grease is held in a colloidal state.

From a physical standpoint, grease in sewage may be classified as capable of floating, when in emulsion or suspension, or of settling. Soap curds and the lighter oils and greases tend to float. Grease in meat-packing wastes may partly sink and partly flow along in emulsion or in suspension, although some of

the material floats. Apparently these types of material tend to rise as the liquid chills. Gas wastes and similar substances may contain relatively heavy tarry residue (which sinks) as well as lighter oils that form a thin film on the surface.

Effects of Grease on Receiving Waterways

When sewage is discharged without treatment into rivers or tidal waters, the normal quantity of grease and oil contained in the sewage tends to give a smooth, silky appearance, called "sleek," to the surface of the body of water near the point of discharge. Small quantities of oil can cover large surfaces and, in a general way, the extent of the sleek field in the vicinity of a given outlet depends upon the grease and oil content of the sewage. Such sleek may be detrimental to boating, due to the formation of oily deposits on the sides of pleasure crafts passing through it. Where sleek fields are sufficiently extensive to reach bathing beaches, the use of the beaches may be out of the question, on account of the deposition of oil and grease.

In the case of rivers, an oil film may interfere with natural purification by shutting off the air needed for re-aeration. Spawning places for fish may be damaged by oil, and fish food injured. Oils may impart unpleasant tastes and odors to water supplies. Furthermore, if the water is used for irrigation, oil films may tend to fill up the pores of the soil.

Another ill effect of discharging untreated sewage into bodies of water is the formation of grease balls. According to Edward Wright, M. Am. Soc. C. E. (3),

> "* * * the amount of fats in the sewage from one of the large tannery districts in Massachusetts, which is discharged into the sea, has been so great as to cause the formation of numberless grease balls, varying in size from a large marble to a cantaloupe. They are particularly offensive in connection with the recreational uses of adjacent waters for bathing. * * * Such balls, presumably from the Boston Harbor outfall sewers, were found at an isolated point 66 miles from the harbor."

Effects of Grease on Sewage Treatment Plants

The troubles caused by grease at sewage treatment plants are many and varied, some of them being of rather common occurrence. In fine-screening plants, grease tends to clog the screens and increase the cost of maintenance. In other types of treatment plants, fatty substances may form unsightly scum on the surface of the sewage in channels and tanks, as well as on the side-walls at the surface of the sewage. In sedimentation tanks, grease may interfere with the settling of suspended solids by coating particles of organic matter and causing them to rise to the surface as scum. In some cases, it has been found that grease digested with difficulty and caused trouble in the digestion chambers of Imhoff tanks.

Fats are a deterrent to the biological processes taking place in intermittent sand filters, trickling filters, and activated-sludge plants, in that they coat with a stable film surfaces that otherwise would be subject to microbial action. They may also cause clogging of filters and of trickling-filter nozzles.

Accumulations of grease are likely to cause offensive odors at sewage treatment plants. They may also destroy paint, as shown by experience at the

Marina sewage pumping plant at San Francisco, Calif. According to Joseph Corrao (4),

> "When oil or grease become emulsified with water they very effectively destroy the paints they come in contact with. They dissolve the vehicle, soften the film and make it susceptible to mechanical damage thereby exposing the steel to attack by water, hydrogen sulfide, and other destructive agents."

Oil and grease in sewage may also cause difficulties in sludge disposal. For example, it was reported in 1931 that, at the Calumet works of the Sanitary District of Chicago, Ill., the activated sludge, which was being heat-dried, showed a marked tendency to ignite in the dryer, when oil was prevalent (5). More recently, at the Southwest Works of Chicago, the distillation of grease and other volatile substances in the flash-drying process has caused accumulations on the forced-draft fans and air ducts, which seriously interfere with the operation of the drying system. Furthermore, large amounts of oil and grease tend to make sludge dewatering more difficult. They depreciate the commercial value of sludge as a fertilizer.

Another possible source of difficulty is the entrainment of large quantities of air in sewage before it reaches the sewage treatment plant. In the early months of 1938, at the Wards Island sewage treatment plant in New York, N. Y., thick scum formed on the surface of the preliminary clarifiers, often to a depth of 4 in. (6). The trouble was traced to the fact that air under pressure was entering the sewer system feeding the plant. Scum formation was eliminated by raising the sewage level at the Manhattan and Bronx grit chambers, thus preventing air from being entrained in the raw sewage.

A difficulty of a similar nature occurred at the North Toronto sewage treatment plant in Toronto, Ont., Canada, where the sewage falls 80 ft in a drop shaft before entering the plant. Large quantities of air, entrained in the raw sewage during this drop, caused the grease in the sewage to form thick froth and scum on the surface of the influent channels. The difficulty was overcome by installing water sprays to beat down the scum in these channels or force it into the preliminary sedimentation tanks.

Tests for Determining Grease in Sewage

The quantity of fats and oils in sewage is commonly determined as ether-soluble matter, by liberating with acid the fatty acids, after evaporating the water, and dissolving them in ether which is then driven off, leaving behind the ether-soluble matter which can be weighed.

Any one of three solvents may be used in the determination of grease, as shown by the following quotation (1):

> "Of the three common fat solvents, petroleum ether, ethyl ether and chloroform, the last extracts the greatest amount of grease, and petroleum ether the smallest. These solvents may extract additional material after the sample has been acidified. This is particularly true of petroleum ether, and to a much smaller extent also true of chloroform. More data are needed regarding the chemical composition of the material extracted by the different solvents, and until this is available it seems best to recommend

petroleum ether, ethyl ether and chloroform. In reporting results the extracting agent should always be stated. On the insufficient data available, chloroform appears to be the most satisfactory solvent. However, it has not received sufficiently widespread use to warrant discarding the other solvents at this time."

Data on the extraction of lipids from sewage sludge by four different solvents have been furnished by O. J. Knechtges, W. H. Petersen, and F. M. Strong (7). The term "lipids" is intended to designate fatty acids, substances yielding fatty acids on hydrolysis, and other substances that are physically of a fat-like character and can be extracted with fat solvents from biological materials. Four solvents were compared with respect to the quantity of lipid extracted from domestic sludge and packing-house sludge. The results (7) are given in Table 1.

TABLE 1.—PERCENTAGE OF DRY SOLIDS EXTRACTED FROM DRIED SLUDGE BY DIFFERENT FAT SOLVENTS
(Sludge Not Acidified)

Sludge	Petroleum ether	Ethyl ether	Isopropyl ether	Chloroform
Domestic	24.7	23.6	31.1	42.5
Packing-house	18.4	21.3	30.3	38.8

Chloroform extracted the largest percentage of material, and petroleum ether the smallest. In fact, without acidification, chloroform extracted approximately twice as much material as did petroleum ether. Even with acidification of the sludge, petroleum ether removed less material than chloroform did without acidification.

Fractionation of the material extracted from five sludges by petroleum ether and chloroform showed that a fraction totaling from 6% to 19% of the extracts was soluble only in chloroform. This fraction was a black, tar-like material with rather acidic properties. The matter was not investigated further, but it was thought that this material might not be lipoidal in character.

TABLE 2.—QUANTITIES OF GREASE IN RAW SEWAGE AT LANCASTER, PA., AS DETERMINED WITH FIVE DIFFERENT SOLVENTS

Solvent	North sewage treatment plant	South sewage treatment plant	Maple Grove pumping station
Chloroform	37.0	147.4	39.2
Petroleum ether	7.9	17.2	12.0
Ethyl ether	31.8	70.0	33.4
Carbon tetrachloride	18.7	74.2	42.2
Benzene (benzol)	23.7	62.6	43.0

Data on the quantity of grease in sewage, as determined by five different solvents, have been furnished by G. J. Wiest from laboratory studies at Lancaster, Pa. (8). Determinations were made on catch samples of raw sewage from the North sewage treatment plant, the South sewage treatment plant, and the Maple Grove pumping station—the sample from the latter point being representative of strictly domestic sewage. In each case the dried solids were acidified before extraction with the solvent. The results of the Lancaster studies on raw sewage are summarized in Table 2. These results show that, in each sample, chloroform extracted a larger quantity of material than petroleum ether or ethyl ether. Furthermore, chloroform extracted more material than

either carbon tetrachloride or benzene, except in the case of the sample of strictly domestic sewage.

The data presented, although rather meager, are sufficient to indicate that the quantity of grease found in sewage by the recommended methods (1) is dependent mostly upon the solvent used for extraction. According to D. A. Okun, Jun. Am. Soc. C. E., and E. Hurwitz and F. W. Mohlman, the literature indicates that chloroform and ethyl ether extract a larger proportion of non-fatty substances than does petroleum ether, which is more specific for fats and oils (9). Chloroform is said to dissolve resins and waxes in addition to strictly fatty substances. It is evident, therefore, that comparisons of results of grease determinations must be made with caution and with due regard for the solvents used in extraction.

The method of grease determination recommended in "Standard Methods for the Examination of Water and Sewage" (1) has not proved entirely satisfactory, aside from the flexibility that it allows in the solvent to be used for extraction. The evaporation of large quantities of sewage required by this method delays the determination considerably and may cause the loss of some of the grease, which is volatile. Furthermore, according to Mr. Mohlman, evaporation to dryness, after acidification to release fatty substances from soaps, may produce low results because of re-saponification of part of the fats on concentration of the solution (9).

To avoid the necessity for lengthy evaporation, as well as possible re-saponification of fats, Richard Pomeroy and C. M. Wakeman have suggested wet

TABLE 3.—GREASE IN THE SEWAGE OF VARIOUS CITIES, AS DETERMINED BY EXTRACTION WITH ETHER

City[a]	Sewage flow (gal)[b]	Grease		Year	Observation period (days)
		ppm	Lb per capita daily		
Alliance, Ohio	131	37	0.040	1920	12
Atlanta, Ga. (Intrenchment Creek plant)	167	104	0.145	1920	11
Baltimore, Md.	106	40	0.035	1920	12
Canton, Ohio	64	62	0.033	1920	9
Cleveland, Ohio (Westerly plant) (12)	...	66	....	1934	196
Columbus, Ohio	94	100	0.078	1920	10
Dayton, Ohio (2)	55	47	0.021	1923	1
Fitchburg, Mass.	88	63	0.046	1920	11
Gloversville, N. Y. (2)[c]	145	489	0.591	1921–22	640
Houston, Tex. (North Side plant)	75	39	0.024	1920	11
Reading, Pa.	101	25	0.021	1920	10
Rochester, N. Y. (Irondequoit plant)	137	34	0.039	1920	12
Toronto, Ont., Canada[d]	142	33	0.039	1931	14

[a] Reference (11) except as noted. [b] Gallons per capita daily. [c] Industrial wastes from tanneries present in large quantities. [d] Data from Toronto.

extraction (10). Their method is to agitate the acidified sample with the solvent and separate the two layers in a separatory funnel. Two extractions complete the removal of the soluble material.

Mr. Mohlman prefers to pass the sample through a specially prepared filter, dry the material retained, and extract with the solvent in a Soxhlet extraction apparatus (9). The filtrate is shaken with solvent, and the solvent, with what-

ever grease it may contain, is then added to the extraction flask. In most cases, especially with sludges, the grease in the filtrate may be disregarded.

Quantities of Grease in Sewage

Table 3 contains data on the quantities of grease in the sewage of various cities, as determined by extraction with ether. Similar data for Massachusetts cities are given in Table 4.

TABLE 4.—Grease in the Sewage of Massachusetts Cities, as Determined by Extraction with Ether

Description (1939 data from the Massachusetts Department of Health)	Attleboro	Brockton	Fitchburg	Gardner		Leominster	North Adams	Pittsfield	Worcester
				Gardner area	Templeton area				
Grease, in parts per million	46	79	45	104	62	43	40	54	80
Period of observation, in days	4	5	7	4	7	7	7	7	5

According to these data, the quantity of grease in city sewage ranges from 25 ppm at Reading, Pa., to somewhat more than 100 ppm at Atlanta, Ga., and Gardner, Mass. In cities where the sewage contains large quantities of industrial wastes, the grease content may be very high. Such is the case at Gloversville, N. Y., where, due to large quantities of tannery wastes in the sewage, nearly 500 ppm of grease were found in the sewage in 1921–1922. From Table 3 it will be seen that, on a per-capita basis, the quantity of grease in sewage ranges from 0.02 lb per day at Dayton, Ohio, Houston, Tex., and Reading, to 0.15 lb at Atlanta, and 0.59 lb at Gloversville.

Additional data on the grease content of sewage have been compiled by W. S. Mahlie, but unfortunately the analytical methods used were not recorded (13).

Methods of Removing Grease from Sewage

There are two general methods of separating grease from sewage: First, by the installation of separate tanks or other devices especially designed to retain grease and other floating matters; and second, by the collection of such materials as they float on the surface of sedimentation tanks or settle to the bottom with sludge. Equipment that may be installed for the first method includes skimming tanks, skimming-detritus tanks, and aerated grease-separating tanks. To aid in the subsequent separation of grease by skimming, the sewage may be agitated with air or by mechanical means, or it may be treated with a combination of air and chlorine.

Plain Skimming.—Probably the earliest method of grease removal at sewage treatment plants was that of skimming the grease by hand from the surface of sedimentation tanks or grit chambers, after arresting the floating grease by means of suitable baffles. In a few cases plain skimming tanks have been installed for the purpose of separating scum from the sewage.

A skimming tank is arranged in such a way that floating matter rises in it to the surface of the sewage and remains there until it is skimmed off, while the liquid flows out continuously through deep outlets or under partitions, curtain walls, or deep scum boards. The few American skimming tanks in existence are either elliptical or circular in shape and furnish detention periods of from 1 to 15 min. The submerged outlet is situated opposite the inlet and at a lower elevation, to promote flotation of the scum and removal of any solid matter that may settle. By means of such skimming tanks material such as corks and bits of wood, as well as grease and oil, is removed.

In a few places, tanks have been built for the separation of the heavy, readily settling solids and of the light floating material in single units, called skimming-detritus tanks. In such a tank at Akron, Ohio, floating wooden booms skim the grease from the surface and collect it in front of an adjustable weir, over which it is pushed on to a draining bed. Revolving plows move the grit and other solids settling from the sewage to a central hopper, whence they are pumped with about 20% of the average sewage flow to grit chambers, where the final separation of grit takes place. After passing fine screens, this "underflow" joins the main body of sewage that has passed through the skimming-detritus tanks. There are two tanks in the Akron plant, 55 ft in diameter and 12 ft in maximum depth, which furnish an average detention period of 12.6 min. There are skimming-detritus tanks of a similar type at Allentown, Pa., and Dayton.

At the West Side works of the Sanitary District of Chicago, the sewage, before entering the Imhoff tanks, passes through rectangular tanks with an average detention period of about 15 min. These tanks are equipped with sludge-removal equipment of the scraper-conveyer type, the return flights on the surface serving as skimmers. About 15% of the flow, containing the settled solids and grit, is diverted to the grit chambers.

Although a considerable quantity of objectionable scum may be removed from sewage by skimming tanks and skimming-detritus tanks, it is doubtful whether such tanks alone are capable of removing more than a small proportion of the grease contained in normal sewage.

Sedimentation and Skimming.—A somewhat larger proportion of the grease is often removed from sewage in sedimentation tanks, especially where such tanks are equipped with skimming devices. In this case, part of the grease rises to the surface and is skimmed off, while another portion settles to the bottom and forms part of the sludge.

In installations equipped with mechanical apparatus for sludge removal, this apparatus, or an attachment to it, may also be used to remove scum. The scraper-conveyer type of mechanism has already been mentioned. Another device adapted to the removal of floating matter is a commercially patented mechanism that consists of a bridge traveling on rails along the longitudinal walls of the tank, with a scraper attached to the bridge by hinged arms. The scraper is used to force the sludge into hoppers at the influent end of the tank and, when raised to the surface of the sewage, it is used to skim the tank.

In tanks where the sludge-removal mechanism cannot be readily adapted for use in skimming, a system of water piping with small nozzles, as used by H. B.

Allen in the Imhoff tanks at Fitchburg, Mass., may be of value. The nozzles—horizontal hack-saw cuts in vertical pipes—discharge jets of water almost horizontally over the surface, driving the accumulated grease and oil to points where they can be collected. An installation of this type in the preliminary sedimentation tanks of the North Toronto plant has given good results.

In some plants grease is removed from the surface of sedimentation tanks by hand. It may be skimmed off by means of dippers, drawn off by lowering inverted sluice gates, or pushed into troughs or bays by squeegees.

Since the determination of fats is not ordinarily made in settled sewage, the available data on the removal of grease from sewage by sedimentation tanks are not plentiful. At the Westerly plant in Cleveland, Ohio, according to the late George B. Gascoigne, M. Am. Soc. C. E., a study of ether-soluble matter was made on 28 weekly composites from January to June in 1934, when the grease catchers were not in operation (12). This study showed the average grease content of the raw sewage to be 66 ppm and that of the settled sewage 46 ppm, with a removal of 20 ppm, or 30%. In experiments on sewage of the Boston Main Drainage system in 1935, the results indicated that about 40% of the 46 ppm of grease present in the raw sewage, as determined by ether extraction, could be removed by skimming and sedimentation in a 2-hr detention period, leaving 28 ppm in the effluent (14). Similar results were obtained by the same investigators in experiments on sewage of the North Metropolitan sewerage system in 1935 and 1936.

At the Sewage Treatment Works of Buffalo, N. Y., determinations have been made of the grease in weekly composites of acidified daily samples of sewage leaving the grit chambers and of the effluent from the sedimentation tanks, by petroleum-ether extraction of solids removed by filtering through absorbent cotton. The conduits, through which the sewage passes to the sedimentation tanks, are aerated to prevent deposition of solids and keep the sewage fresh. The sedimentation tanks have a displacement period of 80 min at rated capacity. They are provided with mechanical equipment for removing the sludge and scum.

TABLE 5.—GREASE REMOVAL BY SEDIMENTATION TANKS AT BUFFALO, N. Y., SEWAGE TREATMENT WORKS DURING 1940

Month	Grease by Petroleum-Ether Extraction of Filtered Solids				Average daily flow of sewage (mgd)
	Grit chamber effluent (ppm)	Sedimentation tank effluent (ppm)	Removed by sedimentation tanks (ppm)	Removal (%)	
Jan.	50.2	42.7	7.8	15.5	127
Feb.	54.7	46.2	8.5	15.6	141
March	53.8	48.8	5.0	9.3	183
April	45.8	43.3	2.5	5.4	174
May	34.4	27.2	7.2	21.2	129
June	37.1	31.5	5.6	15.1	127
July	39.9	23.1	16.8	42.0	118
Aug.	23.4	15.6	7.8	33.3	128
Sept.	38.2	29.7	8.5	22.2	125
Oct.	40.5	24.1	16.4	40.5	113
Nov.	29.8	29.0	0.8	2.8	161
Dec.	23.1	16.9	6.2	26.8	202
Aver.	39.2	31.5	7.7	19.6	144

A monthly summary of data on grease removal by the sedimentation tanks at Buffalo during 1940 is given in Table 5. (Grease content was determined in weekly composites of daily samples acidified with sulfuric acid. The averages were not weighted. The data were furnished by C. R. Velzy, M. Am. Soc. C.

E.) The monthly average grease content of the sewage, as determined in the grit-chamber effluent, varied from 54.7 to 23.1 ppm, averaging 39.2 ppm, for the year, and the proportion of grease removed by the sedimentation tanks varied from 42.0 to 2.8%, averaging 19.6% for the year. There does not seem to have been any definite relation between the quantity of grease in the sewage and the percentage removed.

TABLE 6.—AVERAGE GREASE REMOVAL BY SETTLING TANKS AT SEWAGE TREATMENT PLANTS OF MASSACHUSETTS CITIES DURING 1939

City	Detention period (hr)	Fats by Ether Extraction (ppm)			Fats removed (%)
		Raw sewage	Settled sewage	Removed	
Brockton	2.0[a]	79	56	23	29
Fitchburg	6.9[b]	45	24	21	47
Gardner[c]	....[a]	62	34	28	45
Leominster	1.9[a]	43	27	16	37
Marlborough	6.9[a]	76	34	42	55
North Adams	2.4[a]	40	37	3	8
Pittsfield	3.3[a]	54	15	39	72
Worcester	2.8[b]	80	42	38	47

[a] Sedimentation tank. [b] Imhoff tank. [c] Templeton area.

Data on the removal of grease by settling tanks at sewage treatment plants of Massachusetts cities (15) are presented in Table 6. The data for fats, or grease, are averages for 1939 as determined by ether extraction of the solids of composite samples of raw and settled sewage, collected in most cases at approximately bi-monthly intervals. Table 6 shows that of 43 to 80 ppm of grease in the raw sewage at these plants, from 8% to 72% was removed by the settling tanks. There appears to be no definite relation between detention period and removal of grease at the different plants.

AERATION PRIOR TO SKIMMING

The separation of grease and oil from sewage by flotation is facilitated by a short period of pre-aeration, although, as stated in the report of the Boston experiments of 1935–1936 (14), there is some doubt whether pre-aeration materially increases the over-all removal of grease from sewage by settling tanks. The assistance of aeration in flotation may be due in part to the de-emulsifying of the oil, which is thus separated from the sewage, and in part to physical action, as the grease adheres to the small air bubbles and is carried to the surface with them.

Several systems have been developed for the removal of grease and oil by aeration. Karl Imhoff, M. Am. Soc. C. E., has built combined aerating and skimming tanks at several plants in the Ruhr District in Germany, using detention periods of from 3 to 5 min. Tanks of this type have been installed at Pasadena, Calif., and Springfield, Ill. The tank in the latter city, which was placed in operation in 1931, is 16 ft by 35 ft in plan, with a double row of air-diffuser plates through the center at the bottom and two longitudinal baffles, which form a stilling chamber along each side for the accumulation of grease. The detention period is 8 min at the average flow of 7.5 mgd (16).

At Whittier, Calif., there is an aeration chamber, 40 ft long, 10 ft wide, and 9 ft deep, set in front of a skimming chamber, 10 ft square, in which the scum is collected in a trough while the effluent is drawn off at the bottom. The

aeration system is of the ridge-and-furrow type. At Hamilton, N. Y., there is an aerated grease-separating tank, in which the grease settles in pockets at one end and is removed by means of 2-in. suction pipes (17).

Recently tanks or channels have been installed at some sewage treatment plants, to provide a short period of aeration preceding treatment in sedimentation tanks equipped with skimming devices. After passing through the preaeration tanks, the grease, oil, and other floating matter are collected and removed by the skimming devices within the sedimentation tanks. For example, at the District of Columbia plant the sewage passes through three aerated grease separating tanks, each 15 ft wide, 78 ft long, and 15.5 ft in maximum water depth. Each tank is equipped with two rows of diffuser plates along one side. With all three tanks in operation, the detention period in the grease-separating units is designed to be 4 min at an average flow of 130 mgd. After the grease-separating treatment, the sewage flows into twelve sedimentation tanks with an average detention period of 2 hr, equipped with clarifier mechanisms for sludge removal and skimming baffles for removing scum.

At San Antonio, Tex., and Lancaster, Pa., air-diffuser plates have been installed in the distribution channels leading to the preliminary sedimentation tanks. The sewage is subjected to a short period of aeration and the oil, grease, and other floating matters are skimmed mechanically from the surface of the tanks.

In experiments at Los Angeles, Calif., aeration was found to aid flotation of grease and oil and form a thick, tenacious scum that was easily removed from the grease skimmer (18). Aeration was accomplished by discharging small streams of screened sewage into the inlet channel at high velocity, the nozzles being placed 2 in. above the surface of the sewage. Removals of 50% to 75% of the free grease were obtained with detention periods of from 2 to 3 min.

At Rochelle, Ill., there is a combined grease-separating and flocculation tank, equipped with an "American" downdraft mechanism, in which the sewage is treated with diffused air in the presence of gentle agitation. After passing through this tank, which provides a detention period of 20 to 30 min, the sewage enters two preliminary sedimentation tanks, equipped with skimming mechanisms. A large proportion of the separated grease is skimmed directly from the surface of the separating tank. Removals of grease, determined by ether extraction, ranging from 48% to 69%, have been reported for the grease-separating and primary sedimentation tank units at this plant.

At the Richmond-Sunset sewage treatment plant in San Francisco, the sewage passes through two "grit-grease" tanks—aerated Imhoff-type tanks—which provide a detention period of 10 min (19). The grease is carried by water sprays to a movable trough and thence to a sump. Following the "grit-grease" tanks are two mechanical flocculation tanks with a detention period of 30 min, the effluent from which flows into two sedimentation tanks with a detention period of 1.5 hr at the average flow. Some of the grease is liberated in the inlet channel to the sedimentation tanks. Scum forming on the sedimentation tanks themselves is removed by water sprays. During an investigation in January, February, and March, 1940, comprising 29 tests on day sewage, the removal of grease, as determined by chloroform extraction, ranged

from about 57% for raw sewages containing 115 ppm of grease to about 46% for those containing 60 ppm of grease.

According to Mr. Gascoigne, there were two grease catchers in operation at the Westerly plant in Cleveland during 1933, treating 5 to 20 mgd (12). One of these tanks was of the Imhoff type and the other was similar in design, except

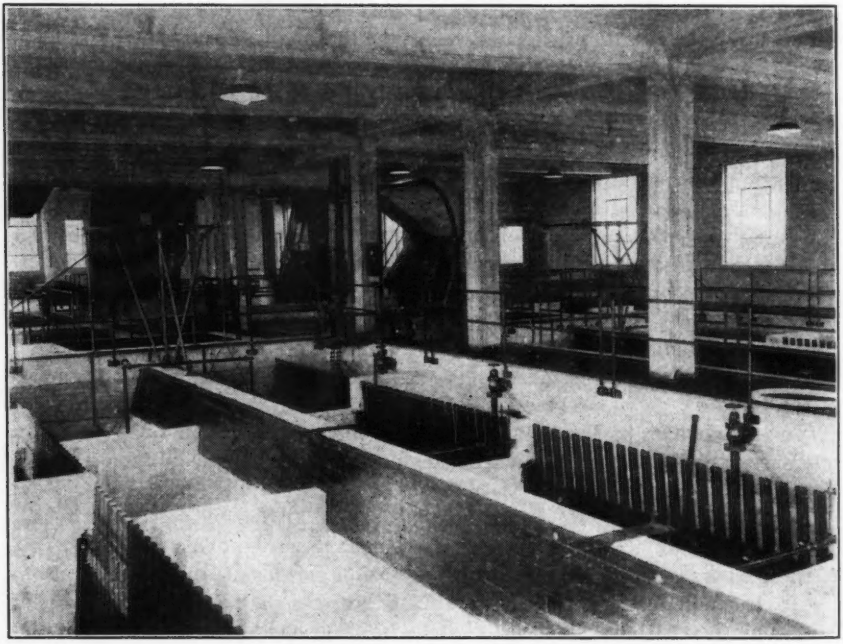

FIG. 1.—AT THE RICHMOND-SUNSET SEWAGE TREATMENT PLANT IN SAN FRANCISCO, THE SEWAGE PASSES THROUGH TWO GRIT-GREASE TANKS

for the inside baffling arrangement. Air was supplied to the influent channels and tanks at a rate of 0.023 cu ft per gal. A study of the ether-soluble matter at this plant for two months in 1933 showed the following results (in parts per million):

Total ether-soluble matter in raw sewage	70
Total ether-soluble matter in settled sewage	46
Removal (34%)	24

This grease was removed in the following forms:

Skimmings from Imhoff tanks	10.5
Skimmings from grease catchers	2.5
Attached to sludge particles	11.0
Total	24.0

Data on the removal of grease at the sewage treatment plant in Dodge City, Kans., during four months of 1932 have been furnished by F. C. Roe (20). At this plant the aeration period was designed to be approximately 13 min. Based on four monthly composite samples, the removal of grease ranged from 44% to 63%, while the quantity of grease in the raw sewage ranged from 25 to 216 ppm.

Aero-Chlorination

To improve the recovery of grease from wool-washing wastes, Jean deRaeve used chlorine (21). He found that the fats were in an emulsified state, protected by a gelatinous nitrogenous colloid, and that chlorine destroyed this protective colloid, allowing the liberated grease to be recovered by flotation and skimming.

An experiment on the use of chlorine as an aid in the removal of grease from sewage was undertaken by H. A. Faber at the sewage treatment plant in Woonsocket, R. I. (22). Here the sewage has a high grease content because it receives wastes from such industries as wool scouring, rayon manufacture, and weaving processes. Gaseous chlorine was injected directly into the compressed air line serving the grease-removal tank. The experiment consisted of aeration with chlorinated air for one week and aeration in the usual manner for the following week. At the end of each 24-hr period the total grease collected in the form of scum on the tank was removed and weighed. The results indicated that a gaseous chlorine dose averaging less than 1.5 ppm effected increased grease removal by skimming, the quantity removed with chlorine and air ranging from 189% to 442% of that removed with air alone.

A group of similar experiments conducted by C. E. Keefer, M. Am. Soc. C. E., and E. C. Cromwell at Baltimore, Md., likewise showed increased grease removals by skimming where chlorine was used. With chlorine injected into the air pipe serving the grease-removal tank, removals ranged from 149% to 847% of the removals when air alone was used (23).

At the South plant in Lancaster, Mr. Wiest tried a form of aero-chlorination under plant-scale conditions in 1938 (24). A chlorinator was set up to feed chlorine in solution directly to the raw sewage just ahead of air diffusion in the pre-aeration channel. The sewage flow was maintained at 4.0 mgd and chlorine was applied at 2.0 ppm. Three tests were run for at least five days each—in the first, neither air nor chlorine was used; in the second, aeration alone was used; and in the third, aero-chlorination was utilized. The results were gaged by determining the grease content of the raw sewage and the effluent from the primary sedimentation tanks, using petroleum ether as a solvent. The removal of grease by sedimentation and skimming during the three tests was as follows:

Description	Percentage removal of grease
Without aeration or chlorination	38
With aeration alone	50
With aero-chlorination	80

During 1939 the aero-chlorination process was placed in operation at the Westerly plant in Cleveland, Ohio, as well as at the sewage treatment plant in Bellefontaine, Ohio. At Dallas, Tex., a treatment plant containing a grease-removal unit was placed in operation in 1940 (25). The grease-removal unit provides a total detention period of 8 min for design flows. During the first 4 min air alone is applied, and the second 4 min consist of aero-chlorination. The ridge-and-furrow type of aeration was selected. The separated grease is swept by flat water sprays to an area of the grease-removal unit, from which it is automatically removed to a scum box at time-clock intervals.

At present (1941) insufficient operating data are available to form a basis for a sound conclusion in regard to the value of aero-chlorination in the over-all removal of grease from sewage.

Other Methods of Grease Removal

For many years sewage treatment plants for residential and institutional sewage have been protected by grease traps, for collecting grease and soap from kitchen and laundry wastes before they are discharged to the sewer. The method is to pass the wastes through a reservoir of sufficient capacity to reduce the temperature of the hot wastes, thus permitting the grease to separate and rise to the surface, whence it may be removed by skimming. During the previous World War, many grease traps were constructed at army camps, where the practice was to clean each trap once a week or every other week. At many of the present National Defense projects, a grease interceptor is being installed for each kitchen sink within the building. The size of the grease interceptor is based on the rate of flow to be passed through it.

The removal of grease from sewage by acid treatment has been practiced at Bradford, England, for many years. Here the sewage contains large amounts of grease due to wool scouring wastes discharged into the sewers. The grease is recovered and put upon the market.

The Miles process of sewage treatment consists of the addition of acid to precipitate solids in the form of sludge which can be dried and degreased, thereby producing for sale both fertilizer and grease. The process has not been adopted in the United States for treatment of domestic sewage, although acid precipitation is used for the treatment of certain kinds of industrial wastes. Very high removal of fats by acid precipitation was reported by R. S. Weston, M. Am. Soc. C. E., and E. S. Dorr in experiments at Boston, Mass., the ether-soluble matter in the sewage being 303 lb per million gallons and in the sludge 430 lb per million gallons (26). This increase was attributed to the decomposition of soaps by the acid. In experiments at New Haven, Conn., C. E. A. Winslow and F. W. Mohlman recorded a removal of approximately 100% of the grease by acid treatment (27).

Disposal of Grease

Grease is removed from sewage in two forms—namely, sludge and scum. Except in the case of acid treatment, the grease which settles with the sludge is disposed of with the latter. Grease removed from sewage by acid treatment may be recovered. At Bradford, England, part of the grease is converted into

such products as soap powders, metallic soaps, lubricants, and pitch (28). Results obtained with a pilot plant led to the design of a full-scale plant, which was under construction in 1938, to handle about one third of the output of raw grease.

Grease and oil removed from sewage in the form of scum may be disposed of by burial, incineration, or digestion with other sewage solids. In some places the oil skimmed from sewage has been used on roads.

The materials collected on the surface of the skimming-detritus tanks at Akron are drawn to small draining beds, and the drained material is disposed of by burial with the dewatered Imhoff sludge. At the plant of the Rahway Valley Joint Meeting in New Jersey, the grease removed from the settling tanks is flushed to a grease-separation chamber, in which the grease is separated from its carrying water by flotation (29). It is then scraped into buckets and buried, although it may be burned in the screenings incinerator. The scum from the skimming-detritus tanks at Allentown and Dayton is incinerated with screenings.

Although troubles of various kinds have been reported, due to the disposal of grease and oil by digestion with sludge (2) (5) (13), the fact remains that it is now common practice to discharge grease, removed from sewage in the form of scum, into separate sludge-digestion tanks for disposal. Among the sewage treatment plants where this method of disposal is used are those for the District of Columbia, and New Rochelle, N. Y.; and the Terminal Island plant at Los Angeles and the Richmond-Sunset plant at San Francisco.

At the District of Columbia plant, where grease is digested with sludge in heated digestion tanks, equipped with fixed covers, no trouble has been experienced in operation. At New Rochelle, where grease is introduced into a heated sludge-digestion tank, equipped with a floating cover, some trouble has been caused by formation of a thick layer of scum in the tank, at one time reaching a thickness of 9 ft. The operator has found that trouble can be avoided by drawing supernatant liquor from a very high elevation at frequent intervals.

At the Terminal Island plant, grease was formerly discharged upon sand beds, dried and burned with the aid of surplus gas from the digester (30). Since this method of disposal proved cumbersome and laborious, it has been superseded by that of discharge into the digester. At the Richmond-Sunset plant, grease is discharged into the sludge-thickening tanks, whence the combined ground screenings, grease, and sludge are pumped into the primary unit of a two-stage sludge-digestion system, equipped with turbo-mixers, heating coils, and a stationary steel dome (19).

Conclusion

The grease content of the sewage is an important consideration in the problem of sewage treatment and disposal.

The quantity of "grease" found in sewage by the present standard method of analysis is largely dependent upon which of the three standard solvents is used for extraction. Differences in the solvent used may lead to confusion when an attempt is made to compare the grease content of sewages of different

cities and, in some cases, that of the sewage of a given city for different years. Furthermore, the standard method of determination of grease in sewage has not proved entirely satisfactory, aside from the choice of solvent which is allowed. Consequently, the following steps appear to be desirable:

(1) Agreement upon an answer to the question as to what substances should be included in the term "grease";

(2) Adoption of a standard method which is most suitable for determination of grease in sewage as thus defined; and

(3) Accumulation of data on grease in sewage, effluents, and sludge based upon this standard method.

The data presented indicate that a considerable proportion of the ether-soluble matter in sewage can be removed by sedimentation and plain skimming. Moreover, the available data show, in general, that a somewhat larger proportion can be removed if the sewage is aerated prior to sedimentation and skimming. Experiments indicate, furthermore, that aero-chlorination prior to sedimentation and skimming accomplishes a still larger removal of grease. At the present time, however, insufficient operating data are available to form a basis for a sound conclusion relative to the value of pre-aeration and of aero-chlorination in the over-all removal of grease from sewage.

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PAPERS

UNUSUAL EVENTS AND THEIR RELATION TO FEDERAL WATER POLICIES

By W. G. Hoyt,[1] M. Am. Soc. C. E.

Synopsis

Within the 10-yr period between 1930 and 1940 there were profound changes in federal water policies and more than a tenfold increase in federal activities designed to modify the runoff phase of the rainfall-runoff cycle. The total federal expenditures for all purposes increased only about two and one-half fold during the same decade. During the fiscal year 1940 alone, some $500,000,000 or $600,000,000 of federal money were spent on measures and practices relating to storage and appurtenant works for irrigation, power, flood control, water supply, water spreading, debris control, recreation, and wild-life refuge; upstream engineering, agricultural, and land-use measures and practices; and all related activities designed to obtain beneficial conservation, control, regulation, and use of water. During 1930 the sum of money so expended was only about one tenth as much. The purpose of this paper is to inquire into the basic reasons underlying the changes in water policies and the large increase in activities and to discuss in a general way their extent, aims, and objectives.

Introduction

Many may feel that the large increase in water conservation activities is directly in accord with the long-established trend toward centralization of governmental responsibilities and corresponding expansion in federal public works. Others feel that the increase has been mainly due to the crossing of a new and largely unexplored frontier and to a normal, though rapid, expansion in long-deferred conservation and wise utilization of soils and waters, which are unquestionably two of the vital natural resources of the United States. Still others feel that social security, which was certainly a major political consideration, was the principal motivating force. All of these explanations may be safely assumed to be valid in some degree.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June, 1942.

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Although recognizing the futility of attempting to evaluate the relative importance of many contributing causes that are obviously interrelated, it would appear that the indicated increase in activities relating to the conservation and control of water can be ascribed largely to the concurrent effect of the following three types of happenings:

(1) The 1930–1940 decade was characterized by a degree of Nation-wide abnormality with respect to climatic and hydrologic conditions that has probably not been exceeded during the past century;

(2) The impact of an economic crisis with chronic symptoms that began in 1929, and the associated depression, brought about changes in political leadership as well as modifications in governmental theory; and

(3) There is a belated Nation-wide recognition that the wastage of soil and water is a genuine menace to the national welfare.

Obviously these three reasons are closely related. For example, the abnormality with respect to meteorologic and hydrologic conditions created the necessity for the rehabilitation of devastated areas, the repair of damage resulting from the unusual events, and the construction activities looking toward alleviation of future occurrences. These in turn provided an outlet for relief work and funds that were made available as a result of the economic crisis. In addition, the visible evidences of the erosion and loss of fertile soil, and its transportation and deposition associated with the widespread floods and with the conditions arising from deficiencies in water supply during droughts, undoubtedly hastened the adoption of federal policies relating to the conservation of soil and water and to the control of floods.

The unusual meteorologic conditions relate largely to droughts and floods. Since 1889 there have been eleven well-recognized drought years in the United States. It is not fully realized, perhaps, that five of these major droughts of record occurred during the 10 years 1930 to 1940, as follows:

Year	Drought area
1930	Practically all states east of the Mississippi River, California, Oregon, Washington, North Dakota, and Montana
1931	Northern Great Plains, Idaho, Utah, Colorado, and the Southeastern States
1933	Upper Mississippi Valley, Nevada and Utah, South Central Plains, and Southeastern States
1934	Practically all of the United States except the Atlantic seaboard, Southern States, and Southeastern States
1936	All of the Mississippi River Valley and northern Great Plains, and Pacific Northwest

The history of these droughts and their significance in relation to climate are so fully recorded in technical reports[2] that it seems unnecessary to do more

[2] "Droughts of 1930–34," by John C. Hoyt, *Water-Supply Paper No. 680*, U. S. Geological Survey, 1936; also "Drought of 1936," by the same author, *Water-Supply Paper No. 820*, U. S. Geological Survey, 1938.

than to list them. They had a decided impact on national affairs and were directly responsible for many new national policies.

Floods, like droughts, also affected all parts of the Nation. Stages and discharges, and resulting damage in basin after basin, broke all historic records.

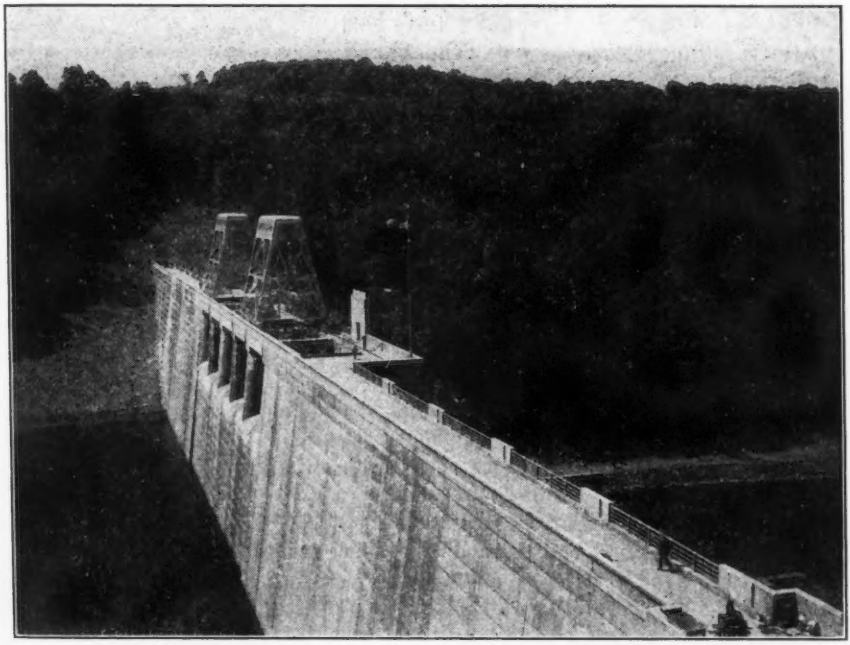

FIG. 1.—FLOODS, LIKE DROUGHTS, ALSO AFFECTED ALL PARTS OF THE NATION. MAHONING CREEK DAM IN WESTERN PENNSYLVANIA IS ONE OF SIX DAMS DESIGNED AS A PART OF THE WAR DEPARTMENT'S COMPREHENSIVE PROGRAM FOR THE CONTROL OF FLOODS IN THE UPPER OHIO RIVER BASIN ABOVE PITTSBURGH, PA.

These flood events occurred generally during the last half of the decade on river basins as follows:

Time	Region
1934:	
January	La Cañada Valley, California
1935:	
May and June	Republican and Kansas rivers in Colorado, Nebraska, and Kansas; and the Nueces and Rio Grande basins in Texas
July	Central New York
August	Muskingum River, Ohio
1936:	
March	All river basins in Northeastern United States from Ohio, West Virginia, and Virginia through to Maine
July and September	Neches, Guadalupe, Concho, and San Saba rivers in Texas

Time	Region
1937:	
January and February	Entire Ohio River Basin
March	Throughout the central upper Mississippi Valley, southern Wisconsin, eastern Iowa, and Illinois
May and June	Canadian and Pecos rivers, New Mexico
December	All of California north of Tehachapi Pass
1938:	
March	All of southern California
September[3]	Hurricane storm wave and flood in New England—believed probably to have destroyed more lives and property than any previous single natural disaster in the history of the United States
1940:	
August	South Atlantic region
November	All of eastern Texas

In addition to the foregoing major flood areas, major and record-breaking floods occurred in scattered areas on smaller streams. Although not absolutely indicative of the unusualness of the past decade with respect to floods, it seems very significant that, out of 6,792 maximum peak discharges covering all available records through September, 1938,[4] 3,137, or nearly 50%, occurred during the 8-yr period ending in 1938. With respect to the March, 1936, flood and the flood of January and February, 1937, in the Ohio River Basin, geologic evidence, suggestive rather than compelling, seems to show that in parts of the area they exceed in height any previous flood. Most of the other floods listed are clearly outstanding and unusual as shown by Myers' scale or other ratings.

It is not surprising to one acquainted with the meteorologic and hydrologic history of, and the resulting damage caused by, these events that the droughts brought home forcefully to the entire Nation the value of water in connection with its economic development or that the cataclysmic flood events made a deeper impression on policy makers, than ever before, of the need for a comprehensive program involving Nation-wide flood control with incidental benefits to power, navigation, irrigation, recreation, and related values.

Although it is somewhat difficult to ascribe definite reasons for the rapid transition of a wide popular movement or belief into an active action program, it is probably fair to state that the drought in 1930 had much to do with the initiation of land-planning and water-planning activities as a distinct federal function; that the recurring droughts during the period 1930 to 1936 resulted in an enlargement of the federal reclamation idea from one of large developments on arid lands to an all-embracing use of water for irrigation of both large

[3] "Hurricane Floods of September, 1938," by C. G. Paulsen, B. L. Bigwood, A. W. Harrington, O. W. Hartwell, and H. B. Kinnison, *Water-Supply Paper No. 867*, U. S. Geological Survey, 1940.

[4] "Maximum Discharges at Stream-Measurement Stations Through December 31, 1941," by Gordon R. Williams and Lawrence C. Crawford, *Water-Supply Paper No 847*, U. S. Geological Survey, 1940.

and small areas in which problems of population shifts and resettlement played an important part for the first time since the elimination of the Nation's geographic frontier; that the widespread disastrous floods resulted in extension of federal flood-control activities from navigable and interstate streams to include others that affect the general welfare, regardless of the size of channel in which they occur, and, what is perhaps more significant, to include so-called "upstream engineering" as an integral part of the flood-control operations; and that the combined occurrences of both unusual floods and droughts resulted in new and extensive federal measures relating to the conservation of soil and water. No one can state definitely whether or not the unusual climatic manifestations would have resulted in new policies and related activities had they not occurred during a period when national affairs were being modified in an effort to overcome an economic depression. The droughts and floods were so unusual that, coming at any time, they would have created an urge, which no doubt would have been translated into definite action programs, at least to some extent, even under more normal economic and political conditions. The only question relates probably to the magnitude of the action program. It is sufficient for the present discussion to indicate that during 1940 hydraulic engineers in federal services were engaged on a far greater number of projects and on a far greater scale than in 1930, and that federal interests in water have grown to enormous proportions during the past decade. This interest has found expression in several different, though related, channels.

National Water Planning

Prior to 1930 there was no single federal agency that devoted itself exclusively to water planning from a national standpoint. Agencies such as the Corps of Engineers, Bureau of Reclamation, and the Geological Survey had collected, compiled, and published a vast amount of basic technical information concerning the Nation's developed and undeveloped potentialities and possibilities. Moreover, just prior to the decade in question, the major floods during the spring of 1927 in the Mississippi Valley and the comprehensive report on the Tennessee River and tributaries by the Corps of Engineers were responsible, in part, for the legislation that resulted in the "308" reports by the Corps of Engineers in which was outlined technical information concerning water problems in all major basins in the United States. In these various agencies, therefore, there was a vast reservoir of technical information that made possible the early initiation of many of the major flood-control and multiple-use projects hereinafter listed.

Early in 1934 the urgent need of large federal public works projects for the unemployment relief program led Congress, on February 2, 1934, to ask the President for a comprehensive plan for the improvement and development of the rivers of the United States that would give information for the guidance of legislation to provide maximum multiple-purpose benefits. Accordingly, the President appointed promptly the secretaries of the Interior, War, Agriculture, and Labor departments to advise on the development of a water policy and on the choice of projects; and on February 20, 1934, they appointed six technical subcomittees, each assigned to a selected geographical region. The Cabinet

Committee (referred to as "The President's Committee on Water Flow") on April 20, 1934, rendered its report to the President with qualifying or supplemental individual statements by the secretaries of War, Agriculture, and Labor. Although this composite report was made in great haste for the primary purpose of determining, quickly, construction projects of highest over-all merit in the unemployment-relief program, it is important in the record of interagency coordination in water planning.

Returning now to consideration of natural phenomena as a major influence, the droughts of the early 1930's seemed to emphasize that in addition to engineering problems it was necessary also to consider their relation to the over-all economic, social, and land problems which had then assumed such great proportions. This situation led to the appointment in 1933 of the Mississippi Valley Committee of the Emergency Administration of Public Works (PWA) which, through a series of administrative changes, became the Water Resources Committee of the National Resources Planning Board. Under the Reorganization Act of 1939 and the Presidential Reorganization Plan No. 4 (April 11, 1940), this Board became a part of the Executive Office of the President, reporting directly to him. It is perhaps significant that the first line in the first planning report—namely, "Report of the Mississippi Valley Committee" dated October 1, 1934—is a quotation credited to Charles D. Norton: "Make no little plans. They have no magic to stir men's souls."[5] No one can gainsay but that this admonition set a keynote and has been adhered to by subsequent committees. Probably no one federal agency has enlisted the services (largely voluntary) of more hydraulic engineers and hydrologists in its planning operations than has the National Resources Planning Board; and many of these men are engaged today on its forty-five drainage basin committees created to facilitate cooperation among federal and state agencies in the conduct of investigations of water in relation to navigation, power, municipal water supply, erosion, irrigation, forestry, floods, droughts, and wildlife conservation. The basin reports of the Water Resources Committee, in which are outlined water problems and programs, contain a wealth of information which has not only been of value in connection with present public works projects but which will increase in value as time goes on. Those interested in hydrology, as such, will no doubt find much of interest in two reports prepared under the auspices of the Water Resources Committee—namely, "Deficiencies in Hydrologic Data" and "Deficiencies in Hydrologic Research."

Flood Control

As has already been indicated, federal (as distinct from non-federal agencies, which had been active in flood-control operations and which include, among others, the Miami Conservancy District, as an aftermath of the 1921 flood, and the Los Angeles Flood Control District) flood activities prior to 1930 were generally incidental to navigation and were conducted largely as a function of the Corps of Engineers. These activities were directed principally toward the control of flood waters by levees and improvement of the channels of navigable or interstate rivers. The cataclysmic flood events of the past decade, however,

[5] Report of the Mississippi Valley Committee of the Public Works Administration, October 1, 1934, p. 3.

greatly modified and enlarged the federal policy and federal participation in relation to flood control in two notable respects: First, by the adoption of a policy which extended the scope of control of floods that affect human life and property, whatever the size of the rivers and channels; and second, by Congressional recognition[6] of a relation between land use and floods and the necessity for a coordinated land-use and water-regulation program for flood control. These two modifications or enlargements of policy not only increased the activities of the Corps of Engineers but also brought the Department of Agriculture "into the picture" as an active flood-control agency. Here again the increased and new activities demanded the effective service of hydraulic engineers. For example, under the Flood Control Act of 1936, Congress authorized some 270 flood-control projects (most of which had been investigated and reported on in the "308" reports), including many dams and reservoirs with an estimated construction cost in excess of $300,000,000 located in thirty-one states and affecting nearly every state in the Union.

The Flood Control Act approved on June 28, 1938, authorized additional flood-control improvements in nineteen separate river basins, or parts of basins, and authorized the sum of $370,000,000 for the construction of the projects included in this Act. In addition to actual construction projects, these acts also authorized the Departments of War and Agriculture to make preliminary examinations and surveys for flood control at some 345 locations. One important feature of the enlarged policy was the expressed obligation for the federal government to assume a larger share of the cost of flood-control operations, which had heretofore been a local responsibility. An outstanding example relates to the reimbursement (Act of June 28, 1938) of the Muskingum Conservancy District for expenditures made by the District in acquiring lands and right of ways for the series of fourteen reservoirs in the Muskingum River Drainage Basin and the inclusion (Act of August 11, 1939) of the reservoirs in the flood-control plan for the Ohio River.

Congressional recognition of a relation between land use and floods and the necessity for a coordinated land-use program for flood control naturally greatly increased flood-control activities in the Department of Agriculture, which since 1938 has been conducting flood-control surveys and developing programs for the reduction of flood damages by the treatment of lands on which floods originate. Preliminary examinations have been completed on more than two hundred watersheds aggregating in area more than 1,000,000 sq miles. Detailed surveys have been initiated on forty-two watersheds having a total area of 265,000 sq miles. Flood-control survey reports had been completed for sixteen of these basins by the end of 1941. Treatment of one of the surveyed watersheds has been initiated. In the Department of Agriculture, especially, the hydrologists have had to play a very important part since it was necessary to evaluate not only the relation between precipitation, land use, and flood runoff but also to predict how these relationships would be modified by changes in land use.

[6] Flood Control Act, June 22, 1936.

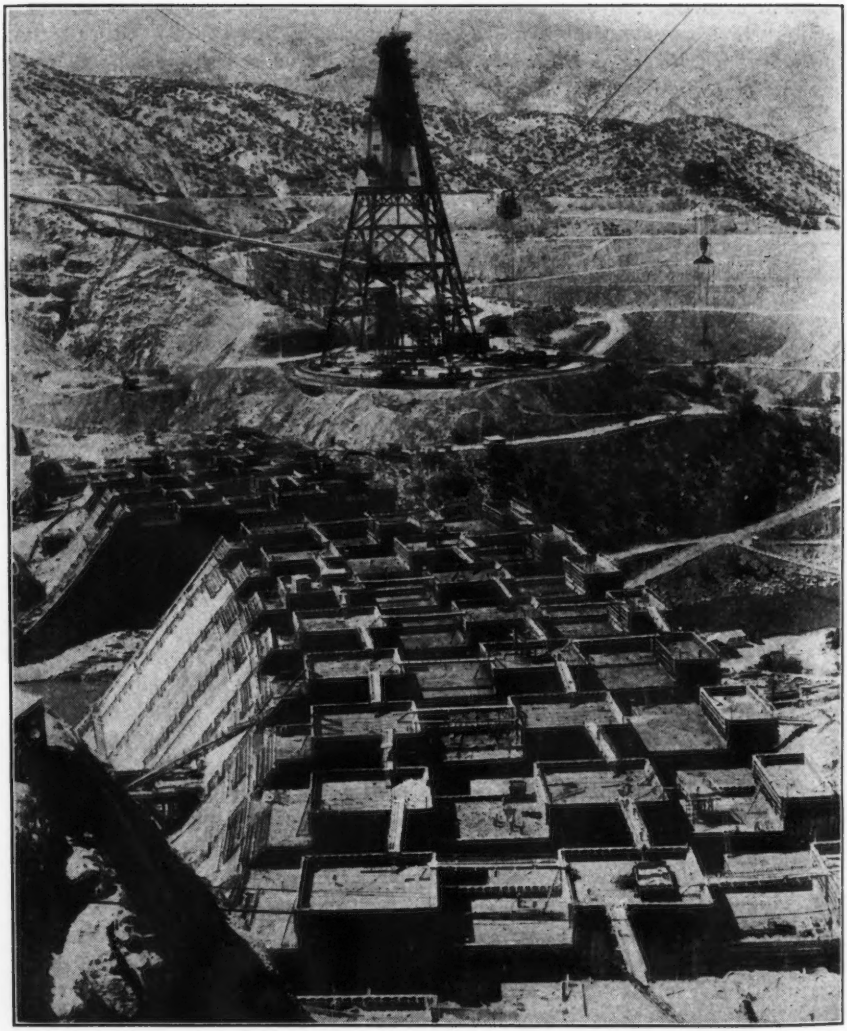

FIG. 2.—NOT ONLY LAND USE BUT POWER DEVELOPMENT WAS TIED IN TO NEW FLOOD-CONTROL PROGRAMS. CRESCENT-SHAPED SHASTA DAM WHICH IS MORE MASSIVE THAN BOULDER AND TALLER THAN GRAND COULEE IS DESIGNED TO MEET THE EVER-INCREASING DEMAND FOR POWER IN THE DEFENSE INDUSTRIES OF NORTHERN CALIFORNIA. THIS DAM, THE PRINCIPAL ENGINEERING FEATURE OF THE CENTRAL VALLEY PROJECT IN CALIFORNIA, WILL ALSO AID IN FLOOD CONTROL, NAVIGATION, IRRIGATION AND THE REPULSION OF SALT WATER FROM THE DELTA FARMING AREA

Water Power

Not only land use but power development as well was tied in to new flood-control programs. The 1936 Flood Control Act permitted the installation of penstocks when the chief of engineers recommended it, whereas the 1938 Act required that such installation be made when recommended by the Federal Power Commission. The latter Act made specific appropriations to be expended by the Federal Power Commission in conducting any necessary investigations or surveys. This step toward federal development of the power by-product was apparently an extension of the power policy of the Federal Water Power Act of 1920, under which all power equipment and appurtenances would be installed by licensees.

Beginning with the Boulder Dam Act of 1928, various projects, including the Tennessee Valley Authority (TVA) developments, have been approved by Congress. This indicates a disposition on the part of Congress to favor actual development by the federal government much more than theretofore. This generalization applies whether the primary purpose be navigation, flood control, or reclamation, although the greatest evidence of the changed policy is disclosed through flood-control measures which, in turn, are the outgrowth of the 1930–1940 flood situation already described.

Multiple-Use Projects

Prior to 1930 the operation of most public water projects related chiefly to a single purpose such as the maintenance of navigation or the storage, release, and distribution of water for irrigation. Since 1930 there has been a profound change in federal participation in multiple-use water projects. A few of these projects, however, represented the consummation of plans proposed prior to 1930 and, therefore, may not be directly related to meteorologic and hydrologic events since 1930. Two such major developments are: The Boulder Dam on the Colorado River, constructed under the Act of 1928, based on earlier reports and examinations, and the Central Valley Project of California, which had its inception in reports by the state engineer of California made prior to 1930. Most of the multiple-use projects, however, have had their inception since 1930 and are more or less related to the train of events since that date. These larger projects are (1) TVA; (2) Fort Peck Dam, in Montana; and (3) Columbia Basin Project.

(1) The all-inclusive TVA (Act of May, 1933) was born in the stress of the swiftly moving days of 1933. The initiation of its chain of navigation, flood-control, and power dams was greatly facilitated by the "308" reports of the Corps of Engineers previously mentioned. Its upstream operations embraced water conservation, erosion, forestry, and land use "in the interests of National defense and for agricultural and industrial development and to improve navigation in the Tennessee River and to control destructive flood waters in the Tennessee River and Mississippi River Basin."[7]

(2) The largest earth dam in the world at Fort Peck, on the Missouri River, was designed to store 19,000,000 acre-ft of water, with its primary engineering

[7] From preamble, Act of May, 1933.

purpose the improvement of river navigation from Sioux City, Iowa, to the mouth of the Missouri River, a distance of 764 miles, with incidental purposes of flood control, hydroelectric power, and irrigation. In addition to the strictly engineering purposes, the dam was initiated to provide work for unemployment. More than 53,000,000 man-hr had been expended on the project to April 15, 1939. For comparison as to the work relief features, it may be of interest to state that the total man-hours by the Civilian Conservation Corps (CCC) directed by the Soil Conservation Service on forestry, soil erosion, flood control, and similar projects from the beginning of operations through December, 1940, was 67,738,591 man-hr.[8]

(3) The Columbia Basin Project and the Grand Coulee Dam on the upper Columbia River, in the State of Washington (the largest single structure ever to be constructed by man and containing enough material to build a concrete road a distance of 5,600 miles), will be used for irrigation, power, flood control, and navigation.

Other projects include the Bonneville development on the Columbia River near Portland, Ore.; the fourteen dams and reservoirs in the Muskingum River Basin, Ohio; the Conchas Dam on the South Canadian River in New Mexico; the Sardis Dam on the Little Tallahatchie River in Mississippi; the Buchanan and Marshall Ford dams on the Colorado River in Texas; the Tygart River Dam in West Virginia; the Santee-Cooper development in South Carolina; and others.

As an example of the magnitude of these and other new great developments, it may be stated that the storage capacity thus made available exceeds 80,000,000 acre-ft, or considerably more than the combined capacity of all reservoirs that had been constructed for all purposes in the United States prior to 1930. The design, construction, and operation of these structures have naturally resulted in vastly increased staffs of the Bureau of Reclamation and the U. S. Engineer Department, the two agencies most actively engaged on the projects. Problems relating to hydraulic engineering and hydrology have been paramount in their design and operation since it was necessary not only that the project works be safe, but that they should effectively perform their multiple-use purposes for navigation, flood control, power, irrigation, domestic water supply, and the real, though intangible, benefits such as recreation, wildlife conservation, National Defense, economic rehabilitation, and the like.

Soil and Water Conservation

Prior to 1930 soil and water conservation activities as a distinct identity separate and apart from federal irrigation, unquestionably the first major federal soil and water-conservation activity, were directed largely toward experimental research and demonstration projects in the Bureau of Agricultural Engineering, Bureau of Chemistry and Soils, and Forest Service in the Department of Agriculture. In the Interior Department, the Geological Survey, since the days of Maj. J. W. Powell, has been active in studies and measures looking toward wise use of land and water resources. Although the need for conservation of soil and water was fully realized by many, the droughts and

[8] The C C.C.'s Eighth Birthday," by G. H. Gilbertson, *Soil Conservation*, SCS, April, 1941, p. 262.

floods provided a stimulus whereby the research activities were translated into a vast action program, first (1933) in the Interior Department under PWA and later (1935) in the Department of Agriculture. The Soil Conservation Act of April 27, 1935,[9] states in its preamble "that the wastage of soil and moisture

FIG. 3.—THE WASTAGE OF SOIL AND MOISTURE RESOURCES RESULTING FROM SOIL EROSION IS A MENACE TO NATIONAL WELFARE (A BASIN LISTED FIELD ARRANGED TO CONSERVE MOISTURE AND PREVENT MOVEMENT OF WATER ALONG THE FURROWS)

resources on farm, grazing, and forest lands of the Nation, resulting from soil erosion, is a menace to national welfare" and declares the policy of Congress

> "to provide permanently for the control and prevention of soil erosion and thereby to preserve natural resources, control floods, prevent impairment of reservoirs, and maintain navigability of rivers and harbors, protect public health, public land and relieve unemployment."

The unusual events also played a large part in the enactment of Soil Conservation District laws in practically every state and the establishment of Soil Conservation districts embracing more than 66,000,000 acres of land, as is evidenced by the fact that the first sentence of a letter by the President, dated February 26, 1937, to the Governors of all states, expressing the hope that state conservation laws would be adopted, reads as follows: "The dust storms and the floods of the last few years have underscored the importance of programs to control soil erosion." Prior to Reorganization Order No. 4, dated April 11, 1940, activities under the Soil Conservation Act were confined to the Department of Agriculture, but under the terms of the Presidential Order No. 4, soil and moisture conservation activities were transferred to the Department of the Interior in so far as about 280,000,000 acres of Indian, grazing, national park, and other lands under its jurisdiction are concerned.

[9] 49 Stat. 163, 16 USC 598.

Closely allied with the purpose of the Soil Conservation Act is the so-called Taylor Grazing Act of June, 1934, which provides for the conservation of soil and water on the vast public domain of the West through the creation and administration of grazing districts. Under this act the Department of the Interior regulates grazing and other forms of land use on some 150,000,000 acres of land that theretofore were subject to entry under the public land laws. Also, under the provisions of the National Industrial Recovery Act of June 16, 1933, all public lands in the eastern and central Western States were withdrawn from entry in February, 1935, by executive order in the interests of conservation of natural resources including "control, utilization and purification of waters, prevention of soil and coastal erosion and flood control."

Thus, since 1930 the Nation has witnessed the expansion of soil and moisture conservation activities as such (and apart from irrigation) from research into a Nation-wide action and educational program by two major federal departments. In addition to providing an outlet for the employment of the CCC, Works Progress Administration (WPA), and similar "work relief" organizations, this development has increased many times the number of hydraulic engineers in the government service.

Irrigation and Water Facilities

Prior to 1930 federal activities in connection with irrigation were conducted chiefly by the Department of the Interior under the Act of June 17, 1902, and amendments thereof, commonly known as the Reclamation Act, and under several specific acts relating to the development of irrigation facilities on Indian lands. Under the Reclamation Act, millions of acres of arid lands of the West had been transformed into productive farms, and substantially all expenditures relating thereto had been on a basis of repayment by water users. Not including Indian lands, most of the projects involved public lands, and economic problems relating to resettlement and population shifts were not generally of primary consideration. Although the federal government was vitally interested in the relation between the adequacy of water to the individual water user on the project, it was not especially concerned with the farmer or rancher on private lands outside of federal projects.

The water shortages throughout the Great Plains, the dust storms, and the human suffering associated with the recurring droughts to which attention has been called led to the passage of three acts, all of which have materially increased federal activity with respect to water. These acts authorized the creation of the now well-known AAA, or Agricultural Adjustment Administration, the Farm Security Administration, and the Water Facilities, all in the Department of Agriculture. Under these acts, investigations, loans, general planning, and home-management guidance, relating to the construction and development of ponds, reservoirs, wells, dams, springs, pumping works, windmills, water spreaders, stock-water tanks, flood irrigation, and small irrigation systems for single farms or small groups of farms, are authorized with not more than $50,000 of federal funds to be expended on a single project.

In addition, the Wheeler-Case Act (August 11, 1939) authorized the Secretary of the Interior to construct water-conservation and utilization projects

in the Great Plains area and elsewhere. This is apparently a new type of conservation legislation involving cooperation between the Bureau of Reclamation, Office of Indian Affairs, Department of Agriculture, federal relief agencies, and local state or political subdivisions, the water users (except Indians, who are dealt with by specific legislation) repaying, over a period of 40 years, all of the allotments from appropriations made for the purpose and, in addition, any amount from "relief funds" directed by the President.

In all these new activities, hydrology has played an important part, since one of the primary purposes of the activities relates to the development of dependable water supplies in regions where the supplies are not only scanty but where, in general, prior development has used the "cream," as it were, of the water. In contrast to 1930, it is now possible under existing laws for any landowner or operator under certain conditions to avail himself of financial and technical help in the development of dependable water supplies for irrigation, stock, and domestic uses.

Hydrologic Research and Fact Finding

As of 1930, federal hydrologic research was restricted generally to a few experimental areas maintained by the Department of Agriculture under the terms of the McSweeney-McNary Forest Research Act or earlier authorization, at which the intricate relationships between rainfall and runoff were determined. The fact-finding activities related largely to the Weather Bureau in connection with the collection of rainfall, snowfall, and temperature records and to the Geological Survey in connection with the Nation-wide collection of records of stream flow, ground water, and quality of water. As a coordinated part of all of the activities previously described, there has been, since 1930, a pronounced increase in hydrologic research activities as a whole, and a substantial, although not proportional, increase in the activities of the Weather Bureau and the Geological Survey as they relate to the collection of basic meteorologic and hydrologic data. The increase in hydrologic research has taken the form, principally, of:

(*a*) Expanded scope and increase in the number of experimental areas maintained by the Soil Conservation Service and the Forest Service for the purpose of determining the intricate relations between rainfall, runoff, soil moisture, and soil movement under different climatic, physiographic, and geologic conditions, and the effect of cover and land use on such relationships.

(*b*) The investigation by the U. S. Engineer Department of approximately 1,000 major storms in the United States involving the compilation of basic precipitation from all available sources; the preparation of isohyetal maps and, for the more important storms, the construction of mass rainfall curves; and computation of duration depth-area curves by 6-hr intervals.

(*c*) Increased activity of the Geological Survey with respect to the analyses of hydrologic and meteorologic phenomena associated with floods and droughts.

(*d*) Expanded hydrologic studies by the Weather Bureau in connection with its enlarged program of flood forecasting and establishment of a hydrometeorological section in which mass rainfall curves prepared by the U. S. Engineer Department are reviewed in connection with synoptic analyses, and in which

special meteorological studies are made to determine limiting precipitation rates in certain drainage basins.

(*e*) Coordinated efforts of the Soil Conservation Service and the Forest Service on hydrologic problems involved in the flood-control activities of the Department of Agriculture.

The increase in availability of basic meteorologic and hydrologic data relates principally to the increase in the number of precipitation stations, more especially the network of recording rain gages installed and maintained by the Weather Bureau in connection with its Nation-wide coverage, made possible in large part by the cooperation of the Corps of Engineers and flood-control agencies in the Department of Agriculture; the dense network in Ohio by the Soi Conservation Service in connection with its microclimatic studies; the increase in snow observations; and the increase in the number of river-measurement stations and ground-water observation wells maintained by the Geological Survey that have been occasioned by the demands of, and made possible by, the cooperation of many federal and non-federal agencies engaged on flood control, irrigation, and related problems. There also may be added numerous fact-finding projects in hydraulic laboratories and experimental areas relating to evaporation, transpiration, wave movement, debris movement, suspended load, and others either independently or jointly by the Corps of Engineers, Geological Survey, Department of Agriculture, Bureau of Reclamation, TVA, and other agencies.

Conclusion

In retrospect it has been made to appear, perhaps, that the water policies outlined were adopted and put into practice in a more orderly sequence and manner than was actually the case. In the war against the depression and unusual climatic and hydrologic conditions of the 1930's, however, as in the present defense activities, there was undoubtedly considerable stress and confusion. Parts of the activities were acknowledged to be experimental. It is too much to hope that under such conditions all policies and practices would ever be intrinscially sound. Future experience may be expected to show that some modifications are needed.

Engineers had considerable to do with the design and planning of projects and also with the formation of policies, although there may be considerable disagreement as to the validity of all the policies. Engineers also have had much to do with the prosecution of the newly authorized activities. Out of their combined experiences should come expressions of opinion and ultimately a knowledge that should have weight in the guidance of future changes and trends. To that extent, as well as in the accomplishment of sound results, the new situation presents a definite challenge to the profession.

Acknowledgment

For the photographic views in this paper the writer is indebted to three important agencies of the federal government. Fig. 1 is supplied through the courtesy of the U. S. Engineer Department; Fig. 2 is from the Bureau of Reclamation; and Fig. 3 is from the Soil Conservation Service.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

REDUCTION OF MINERAL CONTENT IN WATER WITH ORGANIC ZEOLITES

By R. F. Goudey,[1] M. Am. Soc. C. E.

Synopsis

In general, the idea of deliberately reducing the mineral content in water has been limited to the field of boiler-feed water for steam generation purposes. Some incidental reduction in total solids in water has resulted when "temporary" hard waters have been softened with lime for domestic and industrial uses. Modern organic zeolites can now be used to secure an almost complete removal of solids even to the point of competing with distilled water. Already a number of high-pressure steam plants, ice plants, and a single instance of a large munitions plant, are using organic zeolites to produce water of a low mineral content for boiler-feed or rinsing purposes. The use of organic zeolites, while still expensive, has passed the academic stage. It has been recommended (1)[2] that the latest types of organic zeolites, called resinous exchangers, could be used most satisfactorily for processing and precipitate washing at drug and chemical plants.

It is possible to remove one or more of the positive and negative major ions from water including calcium, magnesium, sodium, alkalinity, sulfate, chloride, and nitrates. The full significance of this fact should be realized. The writer suggests that these newer methods of water treatment be considered in the following general instances:

(1) To reduce any major ion in water that exceeds United States Treasury Department Standards, such as sulfates, 250 ppm; chlorides, 250 ppm; and total solids, 1,000 ppm, or the present proposed limit of 500 ppm;

(2) To remove or alter objectionable ions in water for irrigation use with particular attention to lowering the sodium ratio and reducing total solids to within safe limits, even though calcium and magnesium salts can be kept high; and

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June, 1942. Opening discussion on this paper appears elsewhere in this issue.

[1] San. Engr., Bureau of Water Works and Supply, Los Angeles, Calif.

[2] Numerals in parentheses, thus: (1), refer to corresponding items in the Bibliography (Appendix).

(3) To extend the applications in which undesirable waters can be modified for safe industrial use, particularly where relatively high costs can be tolerated.

These suggestions do not recommend the production of distilled water for all uses; but, by blending portions of water variously treated and, in some instances, the addition of compounds, there can be a selective modification of various ions in water to produce a final water satisfactory for the purposes intended. Such a development opens up many practical applications of far-reaching economic importance.

Development of Zeolites

Although the treatment of water with zeolites consists, for the most part, in exchanging calcium and magnesium for sodium, and regenerating the zeolites when exhausted with salt which results in a softer water, there is an actual increase in total solids because the equivalent sodium ions are heavier than the replaced ions. Nevertheless, the treatment by zeolites has been an important stepping stone to the development of more efficient exchange materials which can now be used to remove any of the major ions. The history in the development of zeolites, as applied to water treatment, started with the use of green inorganic sands (glauconite) as early as 1910. This material had an exchange ratio of sodium, for calcium and magnesium, based on about 2,800 grains of hardness (expressed as calcium carbonate) for a cubic foot of material. These natural zeolites were greatly improved during the following decade by processing which doubled the exchange capacity and increased the filtering rate. Common commercial products that were installed at Neville Island, Pennsylvania, in 1926 (2), and at the plant of the City of South Orange, N. J., in 1928 (3), marked the beginning of the use of zeolites for municipal softening.

The first artificial inorganic zeolites were produced as solutions of sodium silicate in 1906, and in the following year the reversible properties of such solutions were discovered. Artificial zeolites are manufactured either from sodium aluminate and sodium silicate to give a ratio of one part sodium oxide to 5.5 or 6.0 parts of silica or from aluminum sulfate and alkalized sodium silicate to give twice the aforementioned ratio. These materials are controlled to form a stiff solid gel or a gelatinous precipitate in liquor. The material is dried either naturally or artificially until it cracks into grains when sprinkled with water. Further control in processing determines the grain size, which is one of the factors in determining the capacity reaction and the loss of head through the filters. Modern inorganic zeolites, therefore, are referred to as "gel-type" zeolites. Their capacity varies from 9,000 to 12,500 grains of hardness removal as calcium carbonate. Most manufacturers guarantee 8,000 grains of hardness removed for each cubic foot of material, although this is not a fair basis for a guarantee as the capacity up to 12,500 grains of hardness removed is dependent on the amount of salt used for regeneration. The rate of filtration for natural zeolites is 5 to 8 gal per sq ft per min with a 40-min

period of regeneration. Artificial zeolites require less rinsing than natural zeolites. Downward filtration may be used with all inorganic zeolites.

Salt consumption for regeneration theoretically should average 0.167 lb of salt for each 1,000 grains of hardness removed. The reaction, however, requires the presence of a salt excess and the actual regeneration dosage under the most advantageous conditions, such as municipal plants, varies from 0.33 to 0.4 lb per 1,000 grains of hardness removed. The amount of salt for regeneration at industrial plants is usually 0.4 to 0.5 lb per 1,000 grains of hardness removed. Even higher salt consumption is allowed for individual domestic units—namely 0.5 to 0.7 lb per 1,000 grains of hardness. The lowest figure for salt consumption requires either reuse of portions of the brine, which is a patented process, regeneration of the zeolite at about 85% capacity, or less, or utilization of longer periods of regeneration.

A large proportion of material used in industry includes solar dried commercial products manufactured at Yuma, Ariz., and Chicago, Ill. Processed material, not solar dried, is manufactured in New York, N. Y., and Chicago. More than forty municipal plants have been constructed since 1935 using zeolites for municipal water softening (4).

A process of adding lime has been developed to remove temporary hardness and to treat a part of the lime-softened water with sodium zeolite to zero hardness for the removal of permanent hardness followed by mixing the raw, lime-treated water and the zeolite-treated water to obtain any final residual hardness desired. Such plants have been located at Findlay, Ohio, and the Metropolitan Water District of Southern California (5).

Lime is the most efficient agent for softening calcium bicarbonate waters because the reaction drops out the lime added and the temporary hardness, except for the solubility value of calcium carbonate, which is usually 40 to 60 ppm. The use of excess lime may remove half this residual. Only zeolites can reduce the calcium and magnesium ions to practically zero. Magnesium bicarbonate hardness requires twice as much lime as calcium, and noncarbonate hardness requires the addition of both lime and soda ash. The sodium zeolites remove these substances with about twice the salt required by the stoichemical reaction, and the advantage of lime and soda ash over zeolites becomes a matter of studying costs of chemicals involved. Usually zeolite softening is cheaper than lime and soda ash when the ratio of noncarbonate hardness is 50% or more of the total hardness (4).

The disadvantages of the sodium zeolite softening are: First, it increases the sodium ratio; second, the sodium zeolite must be used at pH-values between 7.0 and 7.8 as there is disintegration at both higher and lower pH-values; and third, water having passed through a municipal zeolite plant containing a residual hardness and a high pH-value such as 9.0 will cause disintegration of zeolites at local industrial plants which must further treat such waters for complete removal of hardness. Sodium zeolite waters in boiler-feed installations are often treated with sulfuric acid and aerated to remove bicarbonate, but the total solids are slightly increased.

Organic Zeolites.—Following the work of B. A. Adams and E. L. Holmes (6) in 1935, significant progress has been made in the development of organic

zeolites (1). Cation exchangers, or organic zeolites, can be operated either on a sodium or hydrogen cycle depending on whether they are regenerated by salt or mineral acid. Most commercial plants are using 1% sulfuric acid for regeneration purposes. The first four compounds were formerly called carbonaceous zeolites and the last compound is referred to as synthetic resin. Resins are carbonaceous compounds and so the word "organic zeolites" has been selected to cover both phases of development.

The second group of materials which have been recently developed includes the anion exchangers for negative ion removal.

In 1940, the writer presented (7) a short bibliography of the development of carbonaceous zeolites. A more complete bibliography is to be found in a later work by R. J. Myers, J. W. Eastes, and F. J. Myers (1). Artificial organic zeolites have now been developed that are highly condensed, very hard, resistant, and extremely insoluble, and which have properly sized grains with large inner surfaces, high reactivity, and extreme mechanical strength. These zeolites, which are basically either tannin formaldehyde, or m-phenylenediamine resins, have been developed from a great number of compounds. Organic zeolites are somewhat lighter than inorganic zeolites and swell more without rupturing the internal lattice, even to the extent of increasing their size 25% before regeneration. After regeneration the organic zeolites return to normal size. This normal swelling and contracting is probably due to the difference in the size of molecules being exchanged. One third more freeboard for backwashing is required because the material is lighter.

Organic zeolites are installed in filters made acid or alkali resistant with an upward flow rate of 10 gal per sq ft per min with a backwash of 1 to 2 gal per sq ft per min and a rinse of 2 to 3 gal per sq ft per min. The capacity of hydrogen zeolite is proportional to the amount of acid or salt used for regeneration and is proportionately higher for increased capacities. Ordinarily, 0.3 lb of sulfuric acid per 1,000 grains of hardness or its equivalent removed is considered the most efficient method of operation. This represents a removal rate of about 8,000 grains of hardness per cubic foot of material.

In sodium silicate zeolites, the salt ratio is twice the theoretical amount required under the most favorable conditions of operation. Until recently the hydrogen zeolites required 0.292 lb of sulfuric acid per 1,000 grains of hardness removed (7) at a removal rate of 8,550 grains of hardness extracted for each cubic foot of material. This amount of acid is 2.08 times that which is theoretically required. No experiments have yet been published to establish the minimum acid requirements. The writer has used rates as low as 0.225 lb of sulfuric acid per 1,000 grains of hardness removed when operating on calcium only. This is 1.6 times the theoretical amount. That some excess acid must be added is obvious; yet the report of the increased chemical efficiency of resin exchangers used on a sodium cycle as compared with the sodium silicate zeolites indicates that considerably less acid may be required when resin exchangers are operated on a hydrogen cycle and particularly where sodium is high. For purposes of this paper and since the sulfuric acid requirement for treatment is the principal factor, an arbitrary figure of 0.2 lb of sulfuric acid per 1,000 grains of ions removed and expressed as calcium carbonate will be assumed.

Water that has passed through an organic cation exchanger becomes acid and the bicarbonates are converted to carbonic acid gas. The latter can be removed by aeration; hence the cation exchangers remove not only calcium, magnesium, and sodium, but also carbonates and bicarbonates.

The anion or negative ion exchangers have been developed somewhat more slowly. The first exchangers "leaked" and apparently the lattice structure became easily ruptured. Likewise, there was a tendency for the material to go into solution. The present anion exchangers, however, are extremely efficient. They are acid and alkali resistant, have a capacity of 29,800 grains per cubic foot of hardness removed, rinse with about one half the water used formerly, and give twice the filter runs. Regeneration with 1% sodium hydroxide or soda ash is used.

Organic zeolites exchange positive ions with hydrogen when mineral acids are used for regeneration. The exchange is probably on the phenolic hydroxyl group. The equations for initial and final stages are shown in Table 1.

TABLE 1.—TYPICAL EXCHANGE REACTIONS

No.	(*a*) CATION REACTIONS			(*b*) ANION REACTION		
	Hydrogen cycle	Sodium cycle	Magnesium cycle	Hydroxyl cycle	Carbon dioxide cycle	Chloride cycle
	H_2Z	Na_2Z	Mg Z	$Z(OH)_2$	Z CO_2	Z Cl_2
	+	+	+	+	+	+
1	Ca $\{CO_2 (H CO_3)_2$	Ca $\{CO_2 (H CO_3)_2$	Ca $\{CO_2 (H CO_3)_2$	H_2SO_3	2 H Cl	H_2SO_4
2	Mg $\{SO_4$	$\{SO_4$	$\{SO_4$	or	+	↓
3	Na_2 $\{Cl_2$	Mg $\{Cl_2$	$\{Cl_2$	2 H Cl or H_2SO_4	H_2SO_4 ↓	Z Cl_2
	↓	↓	↓	↓	Z $\{Cl_2$	2 H Cl
	Ca, Mg, Na_2 } Z	Ca, Mg } Z	Ca Z		$\{SO_4$	
	+	+	+	Z $\{CO_3$	+	
1	H_2CO_3	Na_2 $\{CO_2 (H CO_3)_2$	Mg $(H CO_3)_2$	Cl	H_2CO_3	
2	H_2SO_4	Na_2SO_4	Mg SO_4	$\{SO_4$ +		
3	2 H Cl	2 Na Cl	Mg Cl_2	2 H_2O		

The removal of negative ions may be due strictly to adsorption or exchange, but for all practical purposes the hydroxide ion behaves parallel with the hydrogen ion and water treated by both methods. The writer found that chlorine replaced sulfates after the hydroxide is exhausted (7). In an experiment of passing 90 gal of Los Angeles (Calif.) aqueduct water through 277 g of a cation exchanger (7) the hydrogen zeolite became successively a sodium and a magnesium zeolite as the capacity to remove sodium and magnesium was reached. This means that hydrogen zeolites may be used to remove calcium only or calcium and magnesium together. Also, 100 gal of cation-treated water without aeration was passed through 333 g of an anion exchanger, and alkalinity, chlorides, and sulfates were removed in the same

order as sodium, magnesium, and calcium. The alkalinity and chlorides exchanged successively with chlorides, finally being replaced with sulfates. This indicates that as ions sulfates can be removed more economically than chlorides. The equations in Table 1(*b*) indicate the initial and final stages of the negative ion reactions.

Regeneration of cation and anion exchangers follows the reactions:
Cation regeneration—

$$\left.\begin{matrix}\mathrm{Ca}\\ \mathrm{Mg}\\ \mathrm{Na_2}\end{matrix}\right\} \mathrm{Z} + \mathrm{H_2SO_4} = \mathrm{H_2Z} + \left.\begin{matrix}\mathrm{Ca}\\ \mathrm{Mg}\\ \mathrm{Na_2}\end{matrix}\right\} \mathrm{SO_4} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

anion regeneration—

$$\mathrm{Zx}\left\{\begin{matrix}\mathrm{Cl}\\ \mathrm{SO_4}\\ \mathrm{H\,CO_3}\end{matrix}\right. + 2\,\mathrm{Na\,OH} = \mathrm{Zx\,OH} + \mathrm{Na_2}\left\{\begin{matrix}\mathrm{Cl_2}\\ \mathrm{SO_4}\\ \mathrm{(H\,CO_3)_2}\end{matrix}\right. \quad \ldots\ldots\ldots (2)$$

Carbonaceous Zeolites Vs. Sodium Zeolites

There is a growing tendency to use carbonaceous zeolites in place of sodium zeolites whether artificial or natural, for the reason that the sodium zeolites require narrow operating limits such as a pH-value from 7.0 to 7.8 beyond which zone there is softening of the structure with subsequent loss of silica. There is very little addition of silica to treated water within safe operating limits. At most small plants, however, the pH-variation is not subject to rigid control and there is, as stated, a tendency to favor a zeolite that is acid and alkali resistant. There are a number of installations using carbonaceous zeolites on a sodium cycle. One of the more recent developments has been the combined use of two carbonaceous zeolite filters (see Fig. 1), one operating with 50% of water on a sodium cycle and the other operating on the other 50% of the water supply on a hydrogen cycle with subsequent mixing of the two waters. There are a number of installations where carbonaceous zeolite is used on a sodium cycle followed by acid treatment to remove alkalinity and thereafter to degasify. This treatment, however, increases the total solids rather than decreases them, although it will produce a water low in hardness and alkalinity.

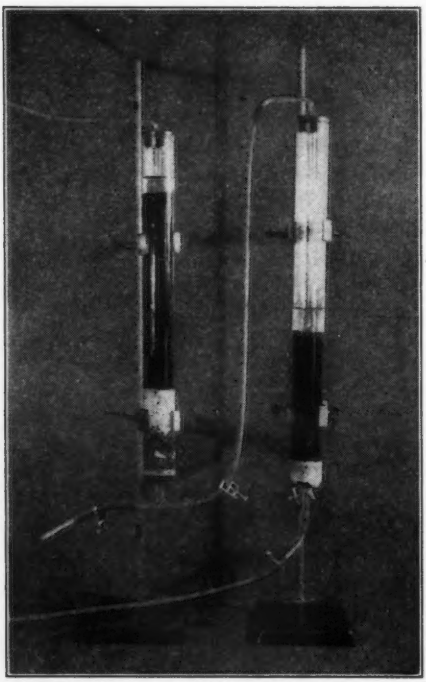

Fig. 1.—Laboratory Carbonaceous Zeolite Filters

Reduction of Solids by Evaporators

In the high-pressure steam plant boiler-feed water field, evaporators have been designed to reduce total solids. With appropriate facilities (including separators so that water can be produced having 4 ppm, or less) total solids and the remaining ions are approximately proportional to the ratio of ions originally present. The first cost is relatively high—namely, about $100,000 per million gal capacity. Operation cost is that of distillation, which is about $2,000 per million gal. Plants that have heat exchange facilities allow practically nothing for the cost of operating evaporators. This process is too limited for general application; yet, where boiler make-up is high, evaporators would be extravagant and hydrogen zeolites offer the most economical treatment.

Cost

Layouts for filters using sodium zeolites have been fairly well established. Municipal zeolite units run from $25,000 to $35,000 per million gal capacity, although the 70-million gal unit of the Metropolitan Water District of Southern California cost $5,000 per million gal. Treatment plants using the hydrogen cycle require greater tank depth but considerably less area and the equipment must be made acid proof. However, there should not be much difference in the final first cost of plants for equivalent treatment capacity. For the removal of alkalinity by aeration, additional equipment is required. Anion exchangers operate at more than twice the rate as cation exchangers so that the cost of filter assemblies is less than that of the corresponding cation exchanger for the same total flow.

The filtering material of exchangers is naturally a major item of expense and the cost of filtering material in any particular case is based on the quantities used. Freight for material shipped cross-country is a large part of the cost where large quantities cannot be manufactured near the points of use. Roughly, the current prices for sodium zeolite in large quantities average $4.50 per cu ft, organic zeolites on the hydrogen cycle at $9 per cu ft, and organic zeolites on the hydroxyl cycle at $36 per cu ft. These prices, while somewhat disconcerting at first glance, are certain to drop downward as use and production increase. From the standpoint of ease in manufacture, there is no reason why ultimately the prices for hydrogen and hydroxyl zeolites should not approach that of sodium zeolites. The royalty fee for hydrogen zeolites is $1 per cu ft, whereas hydroxyl zeolite is $5 per cu ft. Since the foregoing cost range parallels the relative efficiency of the materials involved so far as filtering rate and volume of material required are concerned, the cost of equivalent materials on a million gallon basis of water treated in any new moderately sized plant should be about the same as that now allowed for sodium zeolites. On the basis of present prices, however, the capital investment for moderately sized plants to remove various combinations of ions on a million gallon basis is as shown in Table 2. Obviously, smaller plants might cost 50% more, but the relative ratio of the costs between the various processes is probably the same as given in Table 2. From the standpoint of initial cost, there appears to be no real obstacle to a rather wide

application for the newer processes. The data in Table 2 are based on removing ions to practically zero values. In many instances only a part of the water would be completely treated to be mixed with that ratio of raw water necessary to produce allowable values. If only one third of the water were to be treated,

TABLE 2.—RELATIVE COMPARISON OF INSTALLATION COSTS, IN DOLLARS PER MILLION GALLONS PER DAY CAPACITY, FOR VARIOUS COMBINATION REMOVALS

Description	ION EXCHANGE OR REMOVAL				
	Ca and Mg[a]	+Na	+Alkalinity	+Cl +SO_4	+Cl +SO_4[b]
First cost, thousand dollars per million gallons	35	50	53	70	100
Percentage of total solids removed	30[c]	<50	75[d]	99	99
Cost, in dollars per additional percentage of removal	1,160	750	120	680	1,000

[a] Sodium zeolite. [b] By evaporator. [c] 15% to 40%. [d] 65% to 90%.

the cost per million gallons per day capacity would be one third of that indicated in Table 2.

Operation costs, however, are certain to be high in difficult cases. No general rule can be given owing to wide discrepancies in local prices of sulfuric acid and caustic soda. In July, 1941, the New York price for salt was $13.70 a ton; yet at the plant of the Metropolitan Water District of Southern California, the delivered price at the same time was $4.30 to $4.70 per ton. Sulfuric acid, although quoted in New York at $16.50 a ton, has been known to be delivered in large quantities at $11 a ton. Sodium hydroxide, quoted in New York at $46 a ton, can be delivered as low as $20 a ton at certain locations. In any particular case it would be necessary to calculate the chemical cost with amounts to be used and the cost of delivered chemicals. The amount of acid should be based on 0.2 lb of sulfuric acid per 1,000 grains of ions removed, the latter being expressed in terms of calcium carbonate, and 0.165 lb of sodium hydroxide for 1,000 grains of ions (as calcium carbonate) removed.

PRACTICAL APPLICATIONS

Waters high in bicarbonates lend themselves to high reductions in total solids through cation ion exchanges which, after aeration and mixed with raw waters, are not acid and may not require treatment by anion exchangers. The Los Angeles aqueduct supply is an example of such a water. Waters high in sulfates and chlorides usually require the addition of an alkali to produce a noncorrosive water if both cations and anions are removed in large amounts. The Colorado River water illustrates this type of water. With no thought of recommending the following schemes of treatment, but using them only to illustrate the complexity of reactions and wide variation in costs involved, illustrations follow as to how to produce water with a hardness of 85 ppm, a sodium ratio below 50%, and a sulfate content under 250 ppm, from the two types of waters mentioned.

Case 1. Los Angeles Aqueduct.—In the Los Angeles Aqueduct (see Table 3) only cation treatment is necessary on 23% of the flow which, after aeration, is mixed with 77% of the untreated water. The acidity produced by the calcium, magnesium, and sodium reduction is neutralized by the alkalinity of the mixing water. This treatment requires a sulfuric acid dose of 550 lb per million gal on a chemical cost of $3.00 to $4.55 per million gal, which is relatively low.

It should be observed that were lime to be used to produce such a water, the sodium ratio would exceed an allowable 50%, reaching 53%.

From the standpoint of first cost, this type of treatment for this water is by far the most economical because it would be designed for only 23% of the total flow.

TABLE 3.—EFFECT OF MINERAL ADJUSTMENT
(Parts per Million)

Description	Los Angeles Aqueduct[a]		Colorado River	
	Raw water	Treated water	Raw water	Treated water
Calcium	28	21	125	24
Magnesium	10	8	26	6
Total hardness	111	85	420	85
Sodium	43	33	111	37
% sodium	46	46	36	48
Alkalinity	128	80	136	15
Sulfate	45	45	380	87
Chloride	21	21	92	37
Total solids	252	180	845	200

[a] Cation exchanger only.

Case 2. Colorado River.—Many are the possible combinations of various methods of treatment to bring the Colorado River water within the standards stated. Particular attention must be given to the increasing sodium content as hardness is removed, to reduce sulfates, and to be certain that the final water is not corrosive. Without regard to the cost involved, one logical procedure would be as follows:

1. Treatment of the raw water with excess lime to 20 ppm residual alkalinity;
2. Passing 77% of the water through a cation exchanger to a sodium content of 10 ppm and aerating to remove the increased carbon dioxide acidity;
3. Treating the cation effluent in an anion exchanger to zero sulfates and 20 ppm chlorides;
4. Mixing 23% of the excess lime-softened water with the 77% flow which passed the anion exchanger to give a hardness of 70 ppm; and
5. Addition of 15 ppm of calcium carbonate.

The raw water and treated water analyses would be as shown in Table 3, in which the water has been softened to 85 ppm, with a sodium ratio of less than 50%. In this case the sulfates have been lowered to less than 250 ppm, but that was incidental to meeting the other requirements. The chemicals necessary for such a treatment for a million gallons include 915 lb of lime, 5,500 lb of sulfuric acid, 4,700 lb of sodium hydroxide, and 125 lb of calcium carbonate, which would be in the range of $75 to $150 per million gal depending upon maximum freight and smallness of plant capacity.

Were the Colorado River water not to be softened to 85 ppm, but to 150 ppm or thereabouts, the problem would be considerably simplified.

Case 3. Treatment of Irrigation Water.—There are many instances in which high sodium and sodium chloride waters may be corrected to the ma-

terial advantage of irrigation. For instance, a sodium water of 120 ppm with a 50% sodium ratio and a chloride content which should be reduced by 200 ppm could be treated for $15 per acre-ft. It is realized that this cost is relatively high and would be restricted to the more valuable crops; yet waters of this type determine the entire success or complete failure of important irrigation projects.

The writer is not aware of any location where water is treated to remove minerals where irrigation supplies are involved, although there are a number of instances where good and bad waters are mixed so that the mineral content of the water applied to the land shall not be detrimental for agricultural purposes. In the Southwest it has been generally conceded (8) that the sodium ratio (namely, the percentage of sodium in relation to the total calcium, magnesium, and sodium content expressed in equivalent weights) should not exceed 50% except on well-drained soils where a limit of 60 might be tolerated. Values of 70% or 80% are known to be detrimental for agricultural purposes. The presence of a high ratio of calcium and magnesium salts to sodium salts is apparently beneficial on water used for irrigation purposes. It is also known that when relatively hard waters are treated with sodium zeolites such waters used for irrigation purposes destroy lawns and shrubs. This was observed frequently by the writer at Beverly Hills, Calif., where private zeolite plants have been installed at different residences. There are many irrigation supplies in which either the sodium, chloride, or sulfate content is so high that the water cannot be used beneficially for agricultural purposes. Minor corrections in mineral constituents would go a long way toward making successes or failures of many questionable water supplies. Therefore, in the reduction of mineral contents in irrigation waters, there are many cases in which it would be desirable to reduce the sodium ratio.

Case 4. Boiler-Feed Water.—In the treatment of boiler-feed water, it has been necessary to remove solids to control scale formation and corrosion embrittlement, and to reduce carry-over to a minimum. For quite some time, the high carbonate waters have been treated with acid followed by degasifying so that sodium bicarbonate cannot later break down into carbon dioxide and sodium hydroxide in the boiler. It has been found that the increased cost of making water softening equipment acid resistant, together with the cost of acid, is more than offset in many installations by the reduction in total solids and with the resulting decrease in blowdowns (9).

Each boiler-feed problem depends upon the analyses of the water to determine the best type of treatment. Most cases require a reduction in total solids. The total solids of Los Angeles water can be reduced from 225 to 141 by 50% treatment with sodium zeolites and 50% treatment with hydrogen zeolites. High bicarbonates in the sodium zeolite effluent are used to neutralize the acidity produced by the hydrogen zeolite, and the carbon dioxide is removed by aeration. Obviously, water of any total solid content can be produced using mixtures of raw water with treated portions from cation and anion exchangers.

The cost of reducing the highly bicarbonated Los Angeles water by the sodium zeolite and hydrogen zeolite on a fifty-fifty basis is $15 per million gal and for 50% removal as contrasted with only $20 per million gal for complete removal up to 3 or 4 ppm by use of cation and anion exchanges.

Silica is not removed by positive and negative exchangers and heavy coagulation doses of 300 to 400 ppm of alum may be used prior to the zeolite treatment.

Case 5. Special Cases.—Ice plants desire water with total solids of less than 350 ppm. Beer is best when made with a "plastered" water low in sodium salts. Cartridge cases and washed surfaces, requiring clean and bright faces to prevent jamming, need practically distilled water. Color photography requires water with bicarbonates removed.

Cities with concrete sewers and with sulfates present in the water supply would stop disintegration entirely by removing sulfates. Railroad engine water supplies in the Southwest could be treated better by hydrogen and hydroxyl zeolites. Nuisances and efficiency losses at cooling towers where hydrogen sulfide is produced from mineral sulfates could be controlled by the removal of sulfates.

These and other cases indicate many applications in which solids in water may be reduced to an advantage.

Conclusion

The development of improved organic zeolites for the removal of positive and negative ions, completely or in part, provides superior methods for softening water, reducing sodium content, removing excess alkalinity, lowering sulfates, or reducing chlorides to produce suitable water for domestic, irrigation, and industrial uses, and in special cases where inconvenience, expense, or failure to secure an adequate working plan are involved. Although the costs of operation in the more aggravated cases may be relatively high, they are in most cases not prohibitive and are certain to be lower as increased production and availability of suitable materials develop.

APPENDIX

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

RESISTANCE OF SOFT FILL TO STATIC WHEEL LOADS

By W. Watters Pagon,[1] M. Am. Soc. C. E.

Synopsis

The mathematics of stress in an elastic body, under a load system applied at the surface of the body, is outlined in this paper. The principles are applied to the construction of airplane runways upon a plastic mud fill, to resist airplane wheel loads and to control the placing of granular fill upon the mud. Finally, the paper shows the practical application of the mathematics to the construction of the landing field of the Baltimore (Md.) Municipal Airport.

Historic Background of the Problem

A general history of the Municipal Airport in Baltimore was presented by the writer in 1940, giving in some detail the character of the mud that was pumped into the site as a base for the construction of a landing field, and how this mud fill was allowed to stand for a long period of years before it could be used.[2] Tests indicative of the character of the mud are given in this reference.

At the beginning of the work described herein, about 12,000,000 cu yd of mud, and about 4,000,000 cu yd of granular fill, had been placed by contract. About 400,000 cu yd of the total fill remained to be placed. The granular fill had been hauled by large capacity trucks, either from a distant borrow pit or from barges loaded with dredged material which were moored to the airport bulkheads. It had been placed in the manner most convenient to trucking and, therefore, much of it was in the form of long "fingers" extending into mud areas, the fingers having a depth such that they had been able to support the weight of the heavy trucks.

When it was decided to abandon this method of construction, post-hole auger borings were made on 5-ft centers each way throughout the area of the four landing strips to determine how much granular fill had been placed on the mud along these strips. The depth of fill thus found is shown by the contours in Fig. 1, the 0 contour representing the edge of all areas in which mud was

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June, 1942.

[1] Cons. Engr., Baltimore, Md.

[2] *Transactions*, Am. Soc. C. E., Vol. 106 (1941), p. 1340.

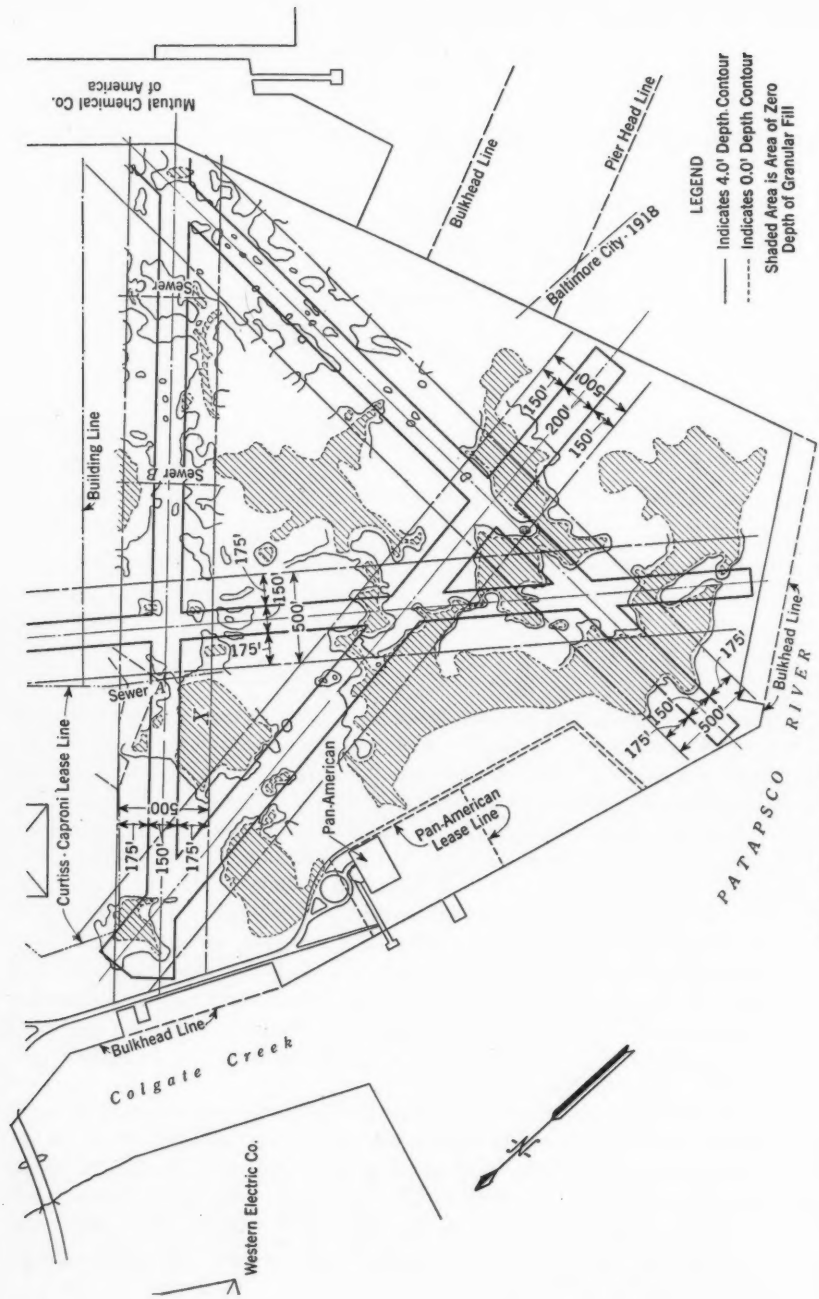

FIG. 1.—DEPTH OF GRANULAR FILL AT THE BEGINNING OF THE WORK

exposed to the view, and the space between the 0 and 4-ft contours representing the areas in which there was less than 4 ft of granular cover. Outside of the 4-ft contour the depth of granular fill varied from 4 ft to more than the 8-ft length of the augers.

Later it was discovered that Fig. 1 is not completely correct. Before the previous contract work was discontinued certain mud areas above airport grades had to be lowered by the use of drag scoops. This grading exposed mud areas that were shallow and that overlay granular fill, or even areas in which the materials were sandwiched together. Nevertheless, Fig. 1 is substantially correct.

In such areas where the mud had bulged above airport grades and had been "decapitated," the consistency was soft, with 12 in. to 18 in. of dried crust, which had formed within a period of about eighteen months. Fig. 2 is a view of

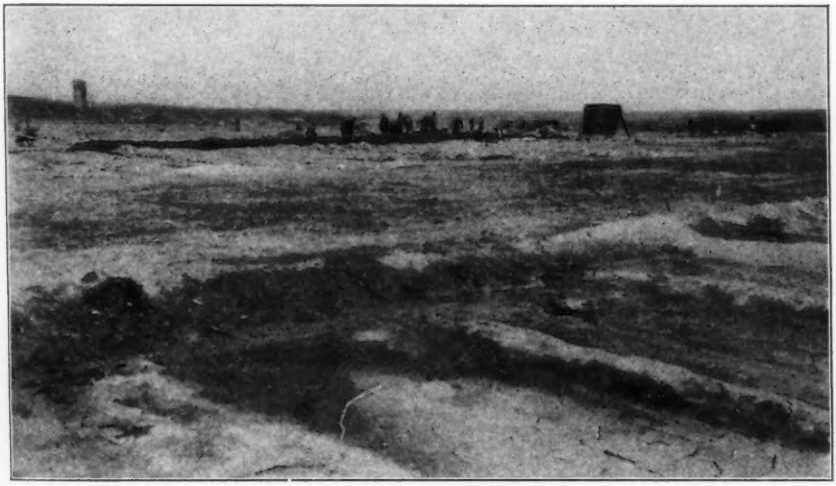

FIG. 2.—CHARACTERISTIC VIEW OF EXCAVATED MUD AREA

such an area. (The cracks shown in the foreground, although similar to the shrinkage cracks of the mud fill, are in a thin layer of mud washed into puddles of rain water and dried in the sun.) Where the old crust had not been disturbed its thickness was greater. Where the mud was near the surface, but covered with a shallow granular fill, there was a firmer crust. Again, the areas at the south side of the airport had much firmer mud than the areas near the north corner, because the discharge line from the dredge which had pumped in the mud about 1928 had been near the south boundary line, and all of the coarser materials had settled out here, leaving the finest materials to progress slowly to the north corner.

Where mud and granular fill are mixed in proper proportions the binding power of the mud produces a very firm product. For example, where mud drippings from trucks (which were hauling it to the disposal areas) became intimately mixed with the granular fill, a wearing surface was formed which

FIG. 1.—DEPTH OF GRANULAR FILL AT THE BEGINNING OF THE WORK

had a remarkably high strength in dry weather. Even in long-continued wet weather only the surface became soft and slick. Again, where fingers of granular fill were quite close together, the mud between had often become much like soft rubber, through which a spade cuts as through cheese.

The Plastic Mud

A "plastic" will be defined herein as a material which is: Perfectly elastic for stresses below some given value, but which is a viscous fluid for stresses in excess of that value. The problem at the Baltimore airport, therefore, involves two phases: (1) The last phase—a completed runway system, constructed of granular fill bearing upon the mud, stabilized at its surface, and sufficient to support airplane wheel loads and to prevent raveling; and (2) the earlier phase, in which it was necessary to superpose the granular fill upon the mud.

The Problem Presented

The construction problem consisted merely of removing the least quantity of mud, and placing a minimum quantity of granular fill—that is, securing a maximum of uniformity in the depth of fill. The depth of fill must be sufficient to prevent weaving of the runway surface under the airplane loads specified by the Civil Aeronautics Administration; the character of fill must be satisfactory for asphalt stabilization because the entire field will slowly settle with time as the water leaves the underlying mud, causing any overlying concrete pavement to break. As far as possible, the material used for fill must be sharp and gritty so as to have a maximum of interlocking capacity and a minimum of capillarity.

The purpose of this paper is to describe the method of determining how great a depth of granular fill should be placed upon the mud, how this fill should be placed, and how the mud should be, and was, removed.

Stress in a Body Produced by a Point Source at Its Surface

The curves of Boussinesq's formula

$$s_z = \frac{3 P z^3}{2 \pi R^5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

are shown in Fig. 3, in which s_z is the stress at various depths z below the ground surface and distances R from the point of application of the load P (r in Fig. 3 is the horizontal component of R). For a depth of 24 in. the stress is about one fourth of that for a 12-in. depth, but for a 36-in. depth it is again more than halved. Of course this theory presupposes a strictly elastic body throughout, whereas the airport consists of a granular and earthy fill overlying elastic mud. Experience has shown that the theory is a good approximation for earth materials, but there is no theory and no experience for the case in which the earth materials are superposed upon a mass of mud. It is fair to assume that so long as the mud is not overstressed, it can be considered an elastic support under a stratum of material that will be pseudo-elastic, or at least capable of transmitting load through a cone of material. That earth materials can so

transmit load was demonstrated at The Pennsylvania State College at State College, Pa.[3]

It can be shown that for a load applied uniformly over a circular plate the maximum shear occurs at a depth about equal to one third of the plate diameter, and has a value equal to about one third of the unit load on the plate. Similarly Horace Lamb[4] shows that for a point load the stresses within the body are compressive for all points that lie within a cone, whose axis contains the line of action of the load, and whose half vertex angle is 68° 32′ (provided Poisson's ratio is 0.25). Therefore, even an uncemented granular fill can be assumed to

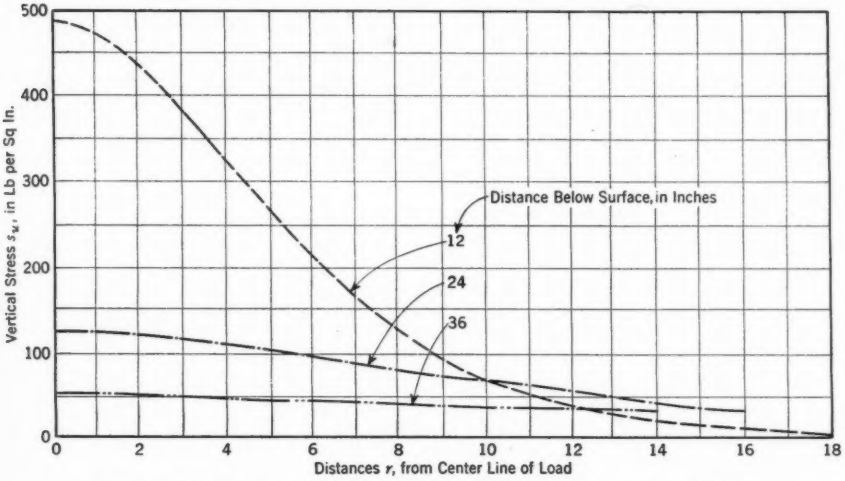

FIG. 3.—CURVES OF THE BOUSSINESQ FORMULA FOR A THOUSAND-POUND LOAD

transmit vertical load to any area, at any depth, whose surface lies within such a cone. For a load distributed over a circular or rectangular plate there will be a frustum of a cone or pyramid within which there will be compression only.

Hence, for want of a theory that expresses Boussinesq's method as applied to a composite body having two layers, each perfectly elastic but with different moduli of elasticity, it becomes necessary to use the cone-of-compressed-material concept, and in the following the half vertex angle has been taken as 60°.

If the computation is based upon an airplane wheel concentration of $P = 50{,}000$ lb, there immediately arises the question of contact area between tire and paving. At present it is true that the maximum tire pressure is about 65 lb per sq in., and to allow for further development 75 lb per sq in. will be used. This means that such a wheel concentration would require 667 sq in. of contact surface—that is, a circle of 29-in. diameter, or an ellipse of 24 by 36

[3] "Experiments on the Distribution of Vertical Pressure in Earth," by R. B. Fehr, *Bulletin No. 11*, The Pennsylvania State College, State College, Pa., 1919, p. 118.

[4] "Hydrodynamics," by Horace Lamb, University Press, Cambridge, 1925, pp. 188–189; also "A Treatise on the Mathematical Theory of Elasticity," by Augustus Edward Hough Love, University Press, Cambridge, 2d Ed., 1906, p. 189.

in., etc. Bomber B-19 is reported to have a wheel load of 82,000 lb on a tire-imprint area of 1,000 sq in.[5] In order not to exceed a safe basis of computation, a diameter of 36 in. will be used which, for a depth of granular fill of 3 ft, and a half angle of 60°, would load an area of 13.4-ft diameter, or 140 sq ft; and, for a depth of 4 ft, an area of 225 sq ft. The respective uniform unit loads on the mud surface, for $P = 50{,}000$ lb, would then be 360 lb per sq ft, and 225 lb per sq ft.

Using the load of 225 lb per sq ft on the surface of the mud crust, and neglecting any value in the mud crust, the maximum shear, from the foregoing statement, would be about 70 lb per sq ft, and would occur beneath the center of the wheel concentration and at a depth of one or more feet below the mud crust. This might place the highest shear stress in the semiliquid mud; hence it becomes necessary to determine whether the mud can carry a shear of 70 lb.

If the shear exceeds the shearing resistance of the mud, movement will occur until the material flows sufficiently to reduce the maximum stress.

Viscosity Test

To determine roughly what shearing strength the mud possessed, a test was made by screwing a duckpin ball to a galvanized pipe and loading the affair.

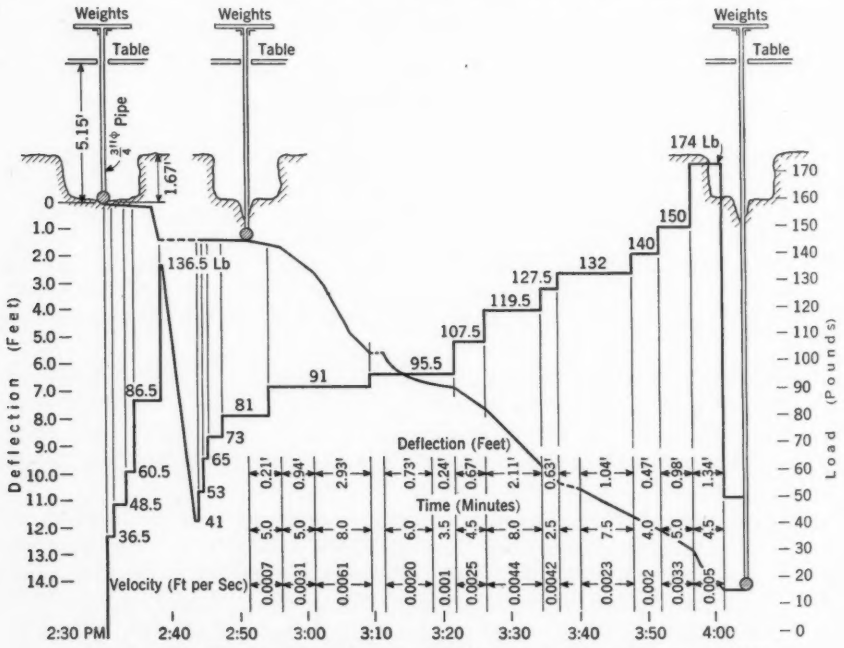

Fig. 4.—Mud Viscosity Test, Baltimore Municipal Airport, October 20, 1939

Fig. 4 shows the curve of this test, with deflections, loading, time, and average velocities of descent. The test could not be entirely continuous, because twice

[5] *Construction Series No. 59*, The Asphalt Inst., p. 5.

it was necessary to add lengths of pipe, which involved removing the load. The ball had a diameter of 4½ in. The site chosen at point *X*, Fig. 1, was an area in which the mud had been decapitated with a drag scoop about 18 months previously, and had then dried in the sun as shown in Fig. 2. The test was started at a depth of about 1.67 ft below the surface, which was where the mud was close to the plastic limit. It was impossible to know what friction was developed on the pipe, but the mud tended to develop a water film on the pipe as it closed in behind the ball, whose frictional resistance would be very small. Therefore, a friction force of 20 lb per sq ft of pipe surface was assumed.

In the deflection range from 2.67 to 5.60 ft, the coefficient of viscosity is about 3,400; from 6.98 to 10.39, about 6,300; from 10.60 to 13.09, 9,700; and from 13.09 to 14.43, 5,900 $\left(\text{units are } \frac{\text{pounds} \times \text{seconds}}{\text{feet}^2}\right)$. If a greater value is given to the pipe friction, these results would differ accordingly. The maximum unit shearing stress on a sphere moving through a viscous liquid is 0.25 p, in which

$$p = \frac{P}{\pi a^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In Eq. 2 P is the load on the sphere and a its radius.

On this basis, with pipe friction deducted, the unit shear in the mud, as the ball slowly descended, varied from 45 lb per sq ft at the bottom of the mud crust to about 60 lb per sq ft at a penetration depth of 14.43 ft, seemingly also indicating an increase of viscosity with depth. These data represent the most extreme condition of fluidity that was known at the beginning of operations, and the results obtained in placing the granular fill confirmed later that this condition equaled the worst conditions found anywhere on the site. In fact, greater difficulty was encountered here (and at one spot still farther north) than anywhere else.

As a result of the test it was felt that present airplane loads of 25 tons per wheel can be carried safely with 4 ft of fill, and for most of the area 3 ft was sufficient, in the writer's judgment. Furthermore, the filling operations should be handled in such a manner that the mud would be stressed higher than 50 lb per sq ft in shear when movement was desired. As the mud dries slowly with time the increasing thickness of the mud crust will permit higher loads to be imposed with safety.

Confirming Load Test

As a practical check on the mathematics, a load test was instituted. A truck was obtained of the low-body type used for hauling large equipment. Its weight was 11,275 lb on two rear axles, each axle being equipped with a pair of dual wheels, and a load of 5,500 lb on the front axle, equipped with two wheels. The wheel base of the rear end is 4 ft 3 in. between axles, and the front axle is 15 ft 10 in. from the forward rear axle. The truck body is 8 ft wide. The rear dual wheels are 12 in. apart, with 5 ft between centers of inner wheels. All wheels are equipped with 9.75 by 20 pneumatic tires. This truck was

loaded with 17 tons of pig lead (see Fig. 5), making a total rear-end load of 22.6 tons. Allowing for tire compression the 22.6 tons were applied on two areas, each about 2 ft by 5 ft or somewhat more, one surrounding each set of four wheels. On actual airplanes having loads comparable to these, the load

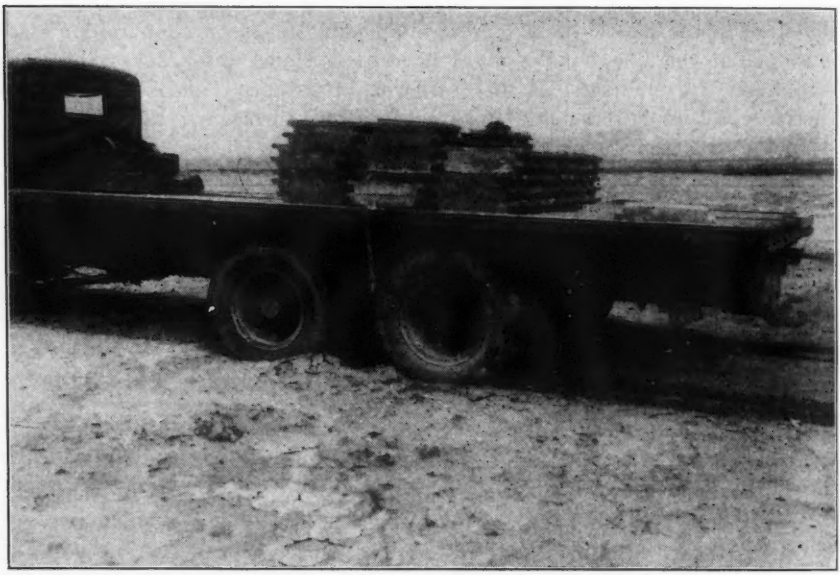

FIG. 5.—TEST TRUCK LOADED WITH SEVENTEEN TONS OF PIG LEAD

wheels are from 18 to 22 ft apart, whereas the two areas combined approximate, roughly, the load on one single airplane wheel group.

This truck was driven about the field where granular fill had already been placed. The most severe condition occurred at one spot where borings indicated a depth of granular fill of only 1.9 ft measured from the surface, and actually several inches less from the bottom of the rut which the tires made in the soft, loose, untreated airport surface (see Fig. 6). In some areas a bull-

FIG. 6.—RUTS FORMED IN THE SOFT SURFACE OF THE GRANULAR FILL

dozer was required to increase the tractive effort. The truck was left in such a position for 24 hr, and showed a settlement of about $\frac{5}{8}$ in. Again at various spots a 47-ton caterpillar crane on a wood mat was operated continuously upon fresh fills of 4-ft depth. When in operation the crane would jar the mud, and

slowly settle, so that every hour or so it was necessary to back it off and place more fill, and then to move it forward for a further reach. The mud clearly is "thixotropic"—that is, it becomes less viscous and more fluid when jarred, but regains its viscosity when allowed to be quiet. The truck-hauling roads also "held up" with a depth of about 4 ft, although in the softest areas there was slow and continuous settlement.

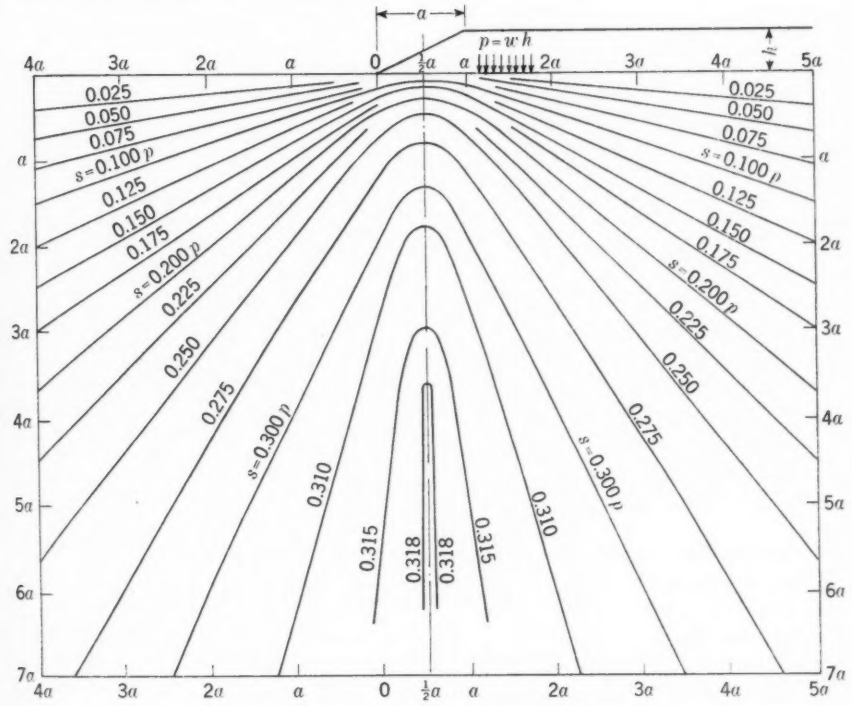

FIG. 7.—SHEAR AT EDGE OF GRANULAR BLANKET

STRESSES DURING CONSTRUCTION OF THE GRANULAR BLANKET

In 1912 S. D. Carothers[6] derived a method of computing stresses within an elastic body when subjected, not to point loads, but to uniform, or uniformly varying, loads, having application only to two-dimensional problems. By assuming certain plane harmonic functions, or combinations of functions, he derived the stress and strain, for example, for a load having a triangular cross section, beginning with zero load along a straight line, and increasing linearly at right angles to this line. If, at a distance a, a reverse load of equal amount is then applied, there results a load condition which represents a uniform fill bounded on one edge by a sloping berm. For instance, this would represent the condition of the airport under a uniform fill advancing across the field with a straight front edge and having a slope along the front edge (see Fig. 7).

[6] "Plane Strain: The Direct Determination of Stress," by S. D. Carothers, *Proceedings*, Royal Soc. of London, Vol. 97, 1920, Series A, p. 110.

By other modifications he obtained the stress condition for an infinitely long prismatic load—as, for example, a highway or railroad fill.

The writer converted this principle to derive the stress condition when a trench of infinite length is opened through the dried mud crust (see Fig. 8). He then amplified this theory by superposing a triangular prism along one or

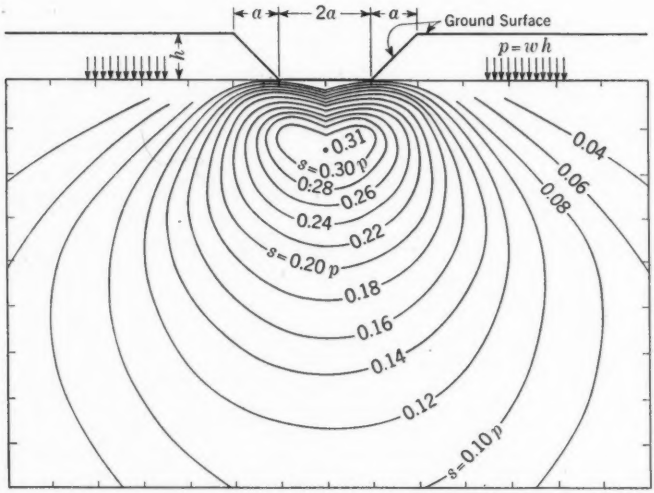

FIG. 8.—SHEAR UNDER A TRENCH

both edges of the trench as shown in Fig. 9. Since shear stress is the condition sought, the shear for different types of loading was computed, and is shown in Fig. 9 in a section at right angles to such loadings, with a sloping berm, having a horizontal width, a, the depth of fill being h, and the weight of fill $w\,h$, or p, per square foot. The drawings are dimensionless; that is, they are proportional throughout to the length a. It will be seen that the shear is maximum, for the blanket, under the middle of the slope, and at a depth of about 3.5 a, with a value of 0.318 $w\,h$. For other relations between the width of side slopes and the width of the trench, the numerical results would change slightly. The maximum shear occurs at a depth approximately equal to a and has a value varying from 0.31 p to 0.35 p. Fig. 10 represents half of the width of a runway, having a crown equal to h, bordered by a trench as shown.

Fig. 11 represents the program that was established for the construction of the runway fills through the soft mud areas. The initial condition is shown in Fig. 11(a), and the final condition in Fig. 11(g). The runway is crowned, and at each edge there is a perforated drain. The depth of granular fill is 4 ft, as is the minimum depth of each lateral drain. Fig. 11(b) indicates the mud area that is subject to motion for the condition shown. The intent was to construct, first, a hauling road down the center of the runway, noted as Fill 2, with (if possible) a hand-placed fill, marked Fill 1, placed previously. As Fill 2 settled under the vibration of the trucks hauling in the material, and dumping

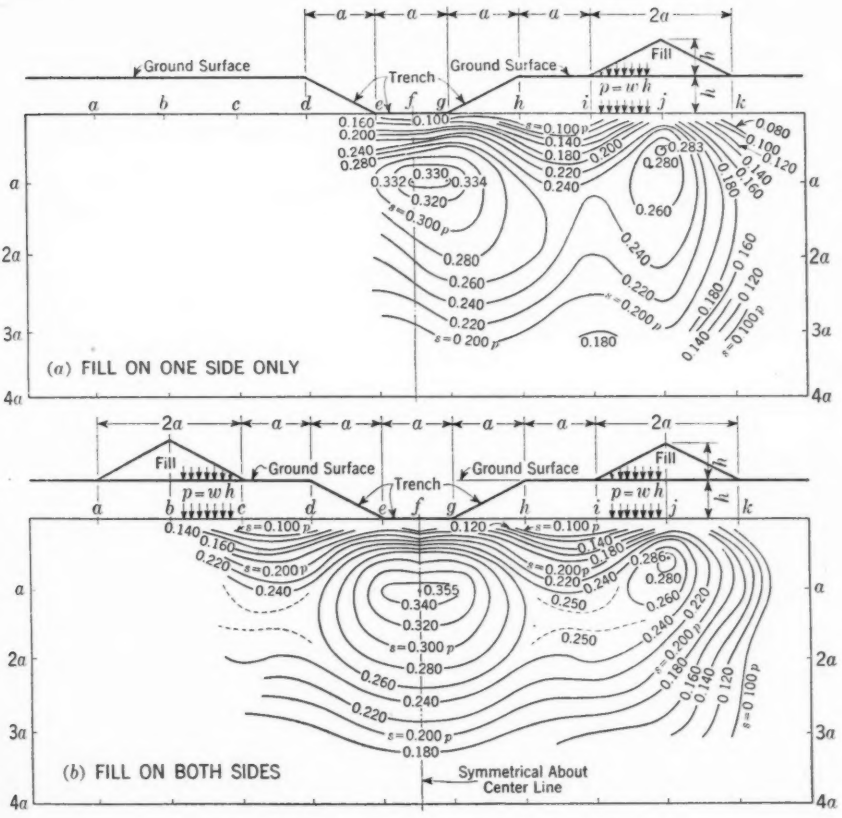

FIG. 9.—SHEAR UNDER A TRENCH, WITH ADJACENT FILL

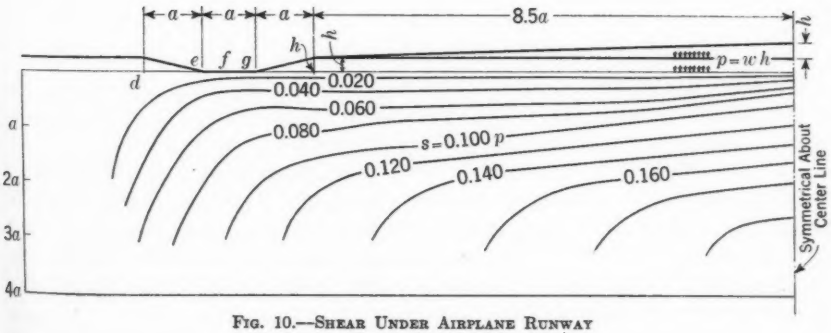

FIG. 10.—SHEAR UNDER AIRPLANE RUNWAY

it along the two sides of the roadway, it would sink until there was a depth of 4 ft. As the trucking operations over Fills 3 caused them to settle, while placing Fills 4, the hauling road would become wider and would have 4 ft of

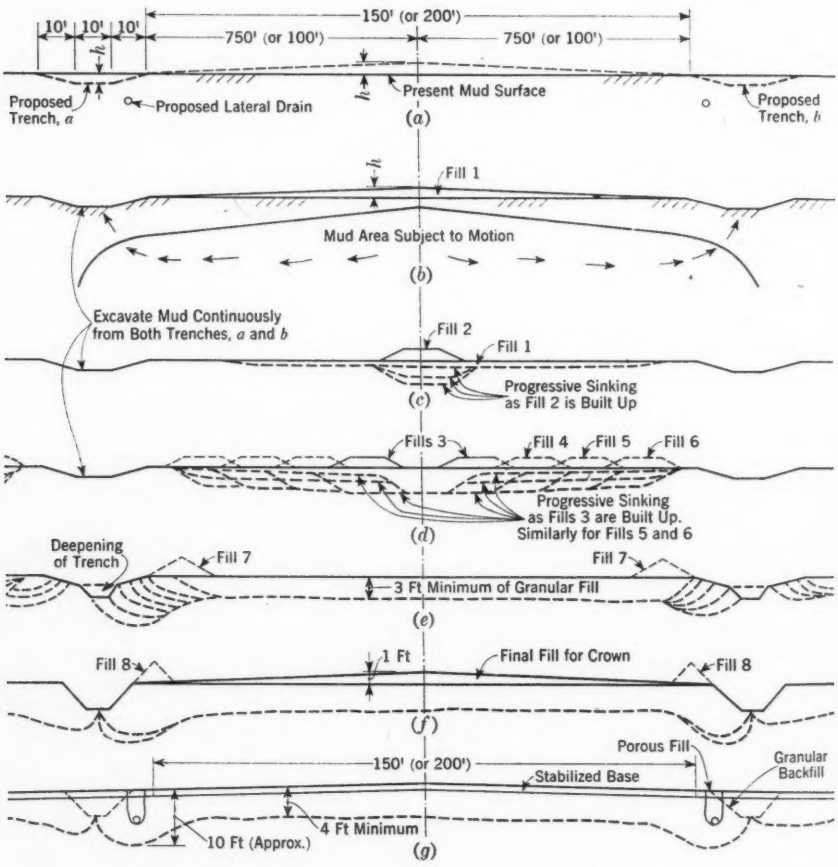

FIG. 11.—PROCEDURE FOR BLEEDING SOFT MUD FROM BENEATH 500-FT LANDING STRIPS

fill beneath it. Only the width of the runway is shown in Fig. 11, but the procedure is the same for the 175-ft (or 150-ft) strips on either side of the runway width. The same description applies to all of the fills up to Fill 7.

To place lateral drains at each edge of the paved runway in the soft mud was impossible, except at great expense for piling. Moreover, there would be no certainty that the drains would remain where they were placed. Hence Fills 7 were designed to extend much deeper, and to pass entirely by the site of the sewer. When they were completed the sewer would then be constructed in an open trench dug into Fills 7.

There was an important additional concept, however, in the trench shown on each side of the runway, outside of the sewers. These trenches, known as

"bleeding trenches," were planned as the line along which mud would be excavated. As the crust would be cut near the same line for the sewers anyhow, two purposes were served at the same time, and the strength of a considerable area of crust thus conserved.

The job was begun as planned, manned almost entirely with hand labor and trucks. The lateral trenches were opened, and the granular fill was brought in along the center line. Where the mud was too stiff to flow into the trenches, triangular fills were placed alongside the trench, so as to squeeze the mud out into it. Of course, there were many spots where the "fingers" of fill placed in the previous operations cut the mud areas so materially that it was necessary to adapt the scheme to local areas. This was done merely by treating an area as a circle, or as several circles, and by excavating the mud from the center and allowing the marginal fill to sink, which it did in the form of a large crater.

FIG. 12.—ONE OF THE BLEEDING TRENCHES AFTER A PERIOD OF RAINY WEATHER

The landing strips have a width of 500 ft, but only the central 150 ft (200 ft for one runway) is paved and underdrained. The treatment of the 175-ft strips outside of the paving was the same, there being an outer trench along the edges of the 500-ft landing strip.

This theory (which is developed from Mr. Carothers' system[6]) is not a theory of viscous flow, but of static stress in shear. Until the test value of 50, 75, or 100 lb per sq ft shearing stress at failure is passed, the mud acts as an elastic body; after this stress has been exceeded, the mud acts as a viscous fluid. Therefore, the criterion is the shearing stress at the weakest point under each load condition. The criterion is indicated by the curves of unit shearing stress. One of the actual trenches is shown in Fig. 12.

After flow begins the mathematics is complex. In many cases the condition can best be described under the caption "isostasy." The granular fill, plus its contained rain water, plus the mud beneath, must balance the mud alone

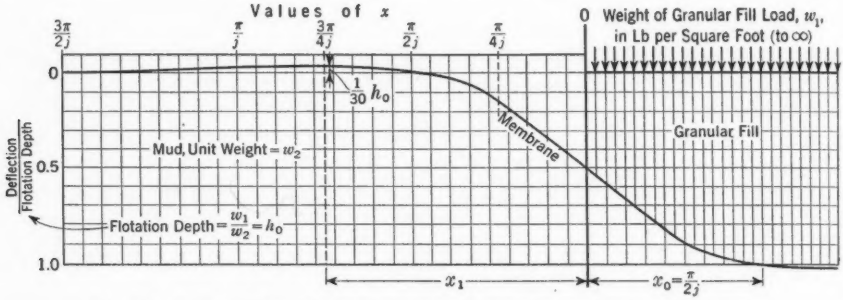

FIG. 13.—FORM OF MUD WAVE, ASSUMING CRUST TO BE AN ELASTIC MEMBRANE OVER A FRICTIONLESS FLUID

in the adjoining areas—having in mind, of course, that there is not exact equality of balance, because viscosity aids unbalance. The result of placing a fill upon an elastic membrane which covers and contains the mud (assumed to be perfectly fluid) is shown in Fig. 13, in which

$$j = \sqrt[4]{\frac{3\,w_2}{E\,h^3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

In Eq. 3, E = modulus of elasticity, w_2 = unit weight of mud, and h = depth of granular fill. When w_2 = 80 lb per cu ft and E = 300,000 lb per sq ft, x_0 for depths of 1, 2, 3, and 4 ft, respectively, is:

h (ft)	x_0 (ft)
1	14
2	24
3	32
4	40

This form of mud wave is quite similar to the waves that actually have developed on the field. It is clear that the shear is a maximum at the edge of the fill. Of course, the dried mud crust cannot be considered a perfect membrane. However, it seldom happened that the crust sheared, so that in effect the crust did form a membrane. At the north corner of the airport the runway grade is 3 ft above the field, and, because of this inclination of the crust as the fill advanced, there were some places where the crust was forced forward until it reached solid fill, when it would at times be forced up over this fill like ice on a lake shore. This was corrected by filling uphill instead of downhill.

In the area near where the ball test was made the mud did not behave as well as elsewhere. Through lack of foresight, and due to the fact that the fill occurred only along one side of the runway, the hauling road was held too long at one site, with several fills made upon it in order to maintain grade. The result was that after a heavy rain storm a considerable waterlogged area sheared

down past the near-by fills and sank a foot or more. It later became stable. At another point, very close to the south side of the field (where the mud in general was stiffest), there was an area of somewhat similar character in which the granular fill subsided, and the mud near-by rose until it stood perhaps 5 ft higher than the granular fill. When such settlements occurred, the soft mud from below would issue slowly through the crack between the two areas, and would "cauliflower" out like the smoke discharge from a volcano, extending at times two or three feet above the surface and slowly drying out. (A discussion of a plastic layer between brittle layers, dealing with the development of salt domes and evidences of motion in the structure of rocks, was presented in 1931 by A. Nadai.[7] For example, the flow of the salt layer under the superincumbent layer of rock is described, and its accumulation at points of weakness to form domes. Photographs of models demonstrate the flow of lava through the throat of a volcano; and they also show slip lines where the plastic materials have sheared on either side of the throat as well as flow lines indicating the character of flow into and up through the throat. At the lower end of the throat the slip lines converge, and the flow lines indicate a sort of "vena contracta," which disappears at a height in the throat of roughly three times the throat diameter, such as is characteristic of viscous flow through a tube, with slip at the walls.)

Method of Construction

As stated, the work was initiated as a purely hand-labor project, having about 1,000 laborers and truckers working in three shifts. The bleeding trenches were first opened up through the mud crust, using runway boards alongside upon which to wheel away the mud in barrows (see Fig. 2). The mud was sticky, which made it difficult to disengage it from the shovel unless a pail of water was handy. The men had to stand on planking. Each day the trench would be excavated to a depth of from 12 in. to 24 in.; each night it would fill up again to within about 6 in. to 12 in. of the surface, the cross section of the surface of the mud as it oozed up being approximately a parabola. On the basis of an unbalance of one foot between the trench bottom and the adjacent mud, and a shear stress amounting to 0.3 $w\,h$, the mud here had a shearing strength of about 30 lb per sq ft, the site having been selected in the area of softest mud. There were many small areas (as shown in Fig. 1) which were bled locally.

The plan of the work is shown in part by Fig. 14 on which is a part of the northwest-southeast runway. The trench layout and the location of the triangular loading fills are shown clearly, as also are the boring depths to the mud beneath the granular fill of the original contract. In the northerly part of this runway the method was successful, but near the southerly end it was soon found that the mud was of almost cheesy consistency and required excavation by means of a clamshell bucket.

The hand-labor method, however, proved to be too slow, and progress was delayed. This was aggravated by the fact that the work was begun in De-

[7] "Plasticity; A Mechanics of the Plastic State of Matter," by A. Nadai, McGraw-Hill Book Co., Inc., 1931, Chapters 43–45.

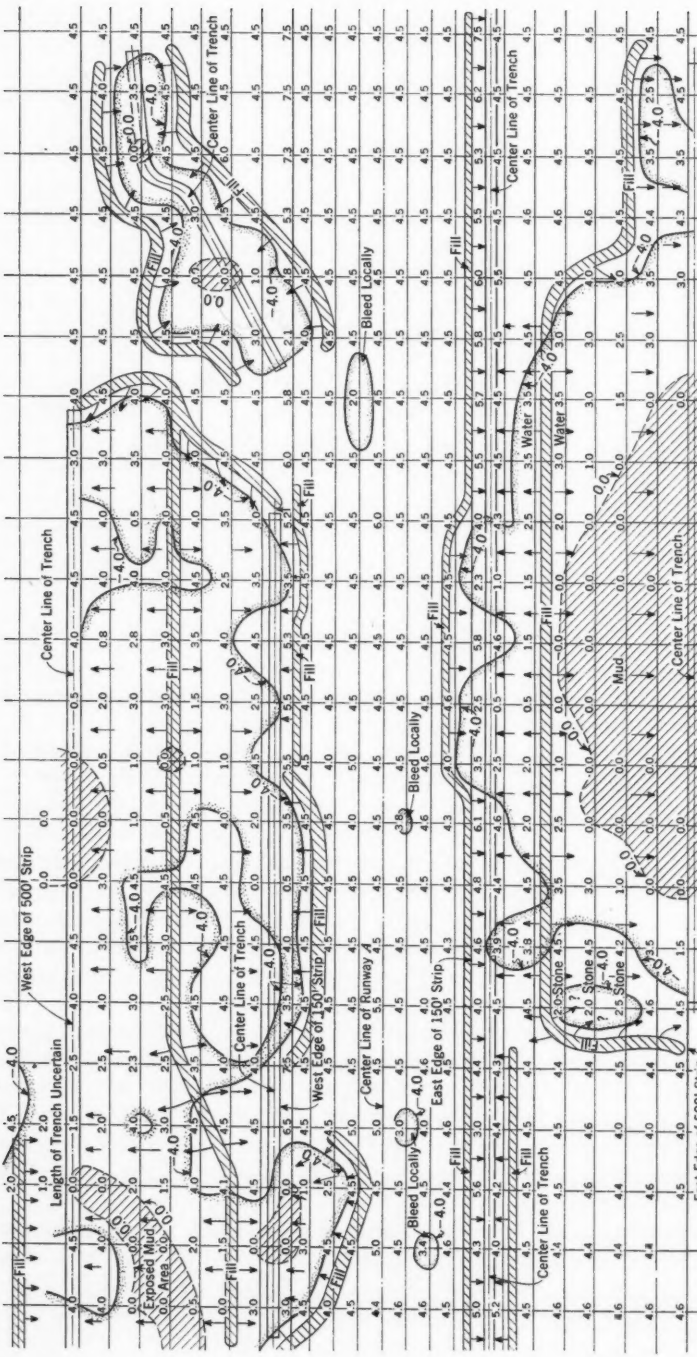

FIG. 14.—TYPICAL LAYOUT OF WORK FOR HAND-LABOR OPERATION

cember, 1939, and the weather throughout the winter was very cold, so that efficient work was impossible. When the engineers decided that the use of cranes would be necessary, the method required modification.

With cranes it was no longer possible to bleed out the mud from the half width of the runway, because the crane booms could not reach sufficiently far. Again it was unnecessary to use the triangular loading fills mentioned previously, because, in a sense, the crane itself, and the trucks serving it, constituted such a fill, and because of excessive vibration. It was desired as far as possible to preserve the dried crust, but a continuous strip operation could not be adhered to everywhere; and in lieu of the continuous strip, a local treatment was adopted. This consisted in forming a loading fill around small cells, by pushing the fill out with bulldozers to form a pocket, and then bleeding the mud out with the crane from a central point of the cell which was within reach of the boom. The net effect of this practice is that the granular fill was placed to reasonably uniform depths, but at the center of each pocket or cell the fill finally closed in upon the surface of liquid mud and not upon dried crust, which means that each such cell will dry more rapidly at its center. In other words, it will dry more rapidly at the point or points from which the mud was bled, and the runway paving, therefore, will develop a sag at each such spot. The principle behind the continuous trench method, as already explained, was to preserve the dried crust over the entire area of the runway paving excepting the narrow strip adjacent to the lateral drains, where no method of construction would preserve the crust.

Lest the impression be gained that the trench method, or the cell method, could have produced a paved area that would settle with perfect uniformity, the writer emphasizes that only in the areas where the mud was exposed over a relatively long distance could the trench method produce uniformity, and there were several such areas having lengths of 500 ft and more. Inspection of Figs. 1 and 14, however, will show that the previous filling operation had left the field in such condition that a large percentage of the area of the runways required local treatments.

FIG. 14.—TYPICAL LAYOUT OF WORK FOR HAND-LABOR OPERATION

FUTURE SETTLEMENT

A most pertinent question must already have formed in the reader's mind—namely, Why should the City of Baltimore undertake to construct an airport upon such a fill? The question was answered by the two consulting engineers employed in Stage Two.[8] The first reason was that the airport had been constructed far beyond the pierhead line of Baltimore Harbor, under a revocable permit of the federal government, and if not used as an airport it was subject to such a "cloud" on the title that it would be unsalable; the second reason was that the city had $3,000,000 invested in it at the time of the engineers' report. Another and more excellent reason was that this site is one of the finest on the Atlantic Coast for a seaplane base, and under the law the landplane base and seaplane base must be contiguous. For such reasons the city was forced to complete the airport, regardless of the character of the mud that had been pumped in.

[8] *Transactions*, Am. Soc. C. E., Vol. 106 (1941), p. 1342.

The consulting engineers stated that the fill would settle for years to come. The mud surface has performed as was predicted, because there was a settlement of the surface of from 2 to 3 ft (in a depth of about 25 ft) in ten years. However, the rate of settlement slows rapidly as the upper part of the mud dries, because the rate of percolation of the water to the surface where it can dry out in the sun is extremely slow, due to the fineness of the mud particles.

Where the dried crust was undisturbed in the operation, the settlement will be slow, for this reason, although it will continue for a long period; where the dried crust was removed, and the granular fill placed upon fresh soft mud, the settlement will be greater, because the upper part of the mud will dry rapidly by absorption of water into the dry fill. This was one of the major reasons why the writer endeavored in every way not to remove the dry crust.

Where the stabilized surface of the runways has been completed for six to ten months the settlement has been sufficiently uniform so that few water pockets have formed on the surface, despite the fact that the crown of the runways is only 9 in. in a width of 150 ft. However, where one runway crosses three main sewers that were built previously upon a pile foundation, and over an old buried concrete bulkhead, there is an evident settlement of nearly 3 in. No other measurements can be made because the surface of the stabilization was not sufficiently exact to permit measurement of the small amount of settlement.

Acknowledgments

A part of the funds expended, and most of the labor required, were provided by the Works Progress Administration, with Lt.-Col. Harry D. Williar, Jr., M. Am. Soc. C. E., as state administrator, and C. F. Bornefeld, M. Am. Soc. C. E., as state director, Division of Operations. The work was done under the direction of George Cobb, chief engineer, and handled directly by Charles F. H. Foard, associate engineer. Sewer construction was under the direction of George E. Finck, M. Am. Soc. C. E., sewerage engineer. Paving and grading were under George A. Carter, highways engineer, and were handled in the field by Charles N. Berigtold, assistant engineer. The writer is airport consulting engineer, and was directly responsible for all mud removal and placing of granular fill, as well as for the buildings and field lighting.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

TECHNIQUE OF DETERMINING SHEARING STRENGTH OF SOILS

PROGRESS REPORT OF A SPECIAL COMMITTEE OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION ON THE TECHNIQUE OF SOIL TESTS

By letter of December 4, 1940, from C. S. Proctor, M. Am. Soc. C. E., to the author, the Executive Committee of the Soil Mechanics and Foundations Division appointed a committee of one to investigate the technique of making soil tests. Shear tests were designated as the first type to be studied and the committee thus far has confined itself to obtaining information on methods in current use for conducting these tests and utilizing the data. The following report presents the information obtained to date from a survey of representative laboratories. The scope of the survey is shown in Table 1. In preparing this report, the committee has supplemented the information obtained from the survey by reference to its background of personal knowledge and experience with the development of soils engineering.

TABLE 1.—SURVEY TO DETERMINE NUMBER OF AGENCIES ENGAGED IN TESTING SOILS FOR SHEAR

No. of laboratories:	Private practice	Universities	Federal agencies	State agencies[a]	Foreign agencies[b]	Total
Canvassed	7	13	13	14	5	52
Replying	5	8	11	10	5	39
Conducting shear tests	5	7	11	2	3	28

[a] State Highway Departments. [b] Canadian and Australian laboratories.

The information requested of each of the cooperating agencies was as follows: (1) A detailed description of the equipment used for shear testing; (2) an outline of testing procedure including sample preparation; and (3) an explanation of methods of interpreting or applying test data to practical problems. The need for information on the first two subjects is believed to be self-evident. The need for the third request may be explained by reference to the fact that the shearing strength of any given soil is not a constant which can be determined independently. Soils resist shearing stresses in

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **June, 1942.** Opening discussion on this Report appears in this issue. Progress Reports are published in *Proceedings* only.

a manner and to a degree dependent on their geological history, on the conditions of loading and drainage, and in some cases on events occurring after load is applied. It is possible, therefore, to have the best testing equipment procurable, to conduct the test in an approved manner, and still to apply the results in such a way that entirely erroneous conclusions are reached. An appraisal of all three aspects of the work is essential, therefore, before an opinion can be reached as to the validity of any soils analysis.

A number of different types of equipment have been used or proposed for measuring the shearing strength of soils. These are translatory devices (direct shear and double shear), triaxial compression (including unconfined compression for cohesive soils), torsional shear on cylindrical and ring samples, rotary shear, triaxial shear on cubical samples, and numerous other devices. Some of these are in use only in research laboratories and some may be found only in Europe. However, it is interesting to note that even in the research laboratories, only the direct shear, double shear, and triaxial compression apparatus were reported in use by the agencies responding to this survey. The basic principles of operation of these three types are represented diagrammatically in Fig. 1.

The details of equipment construction, and the methods of applying loads and measuring reactions and deformations of the specimen, vary considerably in different laboratories. The load may be applied as dead weight, through levers, by a screw device, or by liquid or air pressure, and it may be measured by dead weight, weight on levers, spring balances, proving rings, calibrated bellows, calibrated water tanks, manometers and Bourdon gages, or by any combination of any of them.

Practically all of these shearing devices must be used in the laboratory, although several agencies reported the use of equipment that tests the soil in place. Several of the devices are designed to test the soil without additional rehandling; that is, soil specimens are obtained in seamless liners which are then placed directly in the testing apparatus. These machines are usually of the double-shear type, although one agency uses this system for a direct-shear test.

The first two types (namely, the direct-shear and double-shear devices, often referred to as translatory devices) are constructed on the principle of controlling and measuring, directly, the normal and tangential stresses on predetermined planes of shear in the specimen. In this respect, they differ from the triaxial shearing device in which loads are applied to the specimen with the intention of varying the minor and major principal stresses so that, on some initially undesignated plane, the specimen will fail in shear. Of these two different principles of operation the first seems more convenient when it is desired to preconsolidate the specimen in such a way that the consolidating load subsequently becomes the normal on the plane of shear. In general, however, one method of applying loads does not appear to have any great advantage over the others. There is somewhat more opportunity for comment on the mode of shear. The translatory devices restrict the location of the failure plane to one or two positions in the specimen. In the triaxial equip-

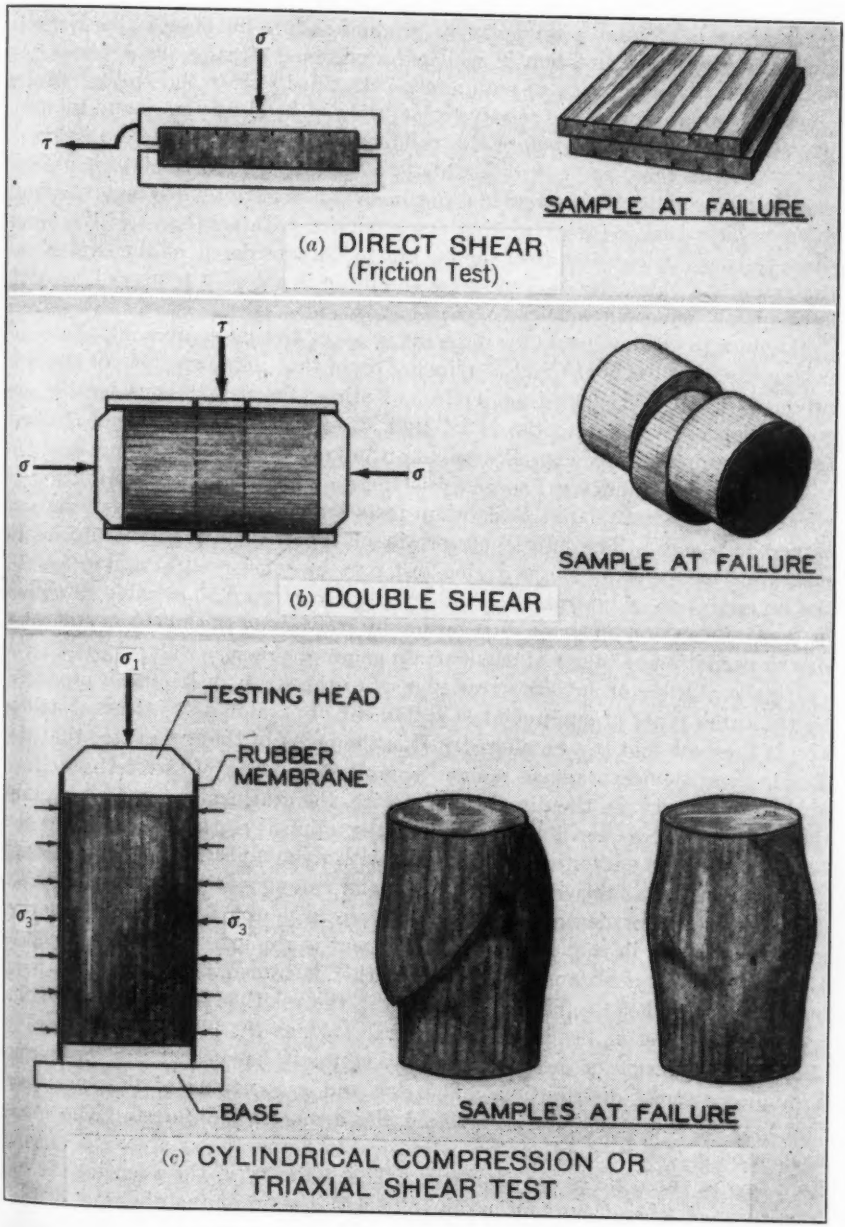

FIG. 1.—CYLINDRICAL COMPRESSION OR TRIAXIAL SHEAR TEST

ment, although the constraint on the location of the failure plane is not as great, there is at least a tendency to produce failure on planes at a definite inclination with the direction in which the principal stresses are applied. In either case, the orientation of nonuniform, stratified soils in the testing device must be compared with the relative orientation of bedding planes and induced stresses in the prototype before the validity of test results can be appraised.

In general, there are two different types of tests—those that seek to produce known conditions of stress in a specimen and those intended only to afford some empirical basis for comparing one specimen with another. If the three shear-testing devices described herein are to be considered in the first class, the extent to which the desired stress condition is actually produced by each device must be considered.

Despite the claims made for different types of testing equipment, the committee has been unable to find convincing proof that a known state of stress is produced in any shear-testing device. Rather, there is a considerable accumulation of evidence to the effect that stress conditions produced in most types of equipment are complicated to an unknown extent by the manner in which loads are applied. Especially is there doubt as to the distribution of effective or intergranular stresses when tests are performed on two-phase and three-phase soils. The efforts of certain of the cooperating laboratories to measure pore-water pressures during test may be considered as a step toward the investigation of internal stress distribution. It may be possible to obtain further information on stress distribution by testing soil specimens of sufficient size to permit embedment of small strain gages or pressure cells.

In the absence of definite knowledge as to the stress distribution produced by the three types of equipment, a statement of opinion as to stress distribution is the best that can be offered. It is the belief of the committee that the double-shear device produces a more complex condition of stress than either of the other two. In the direct-shear device, the gratings or teeth which grip the sample tend to distribute the externally applied shearing loads over the failure plane in the interior of the sample with some uniformity. This feature is lacking in the double-shear equipment and consequently the interior stress distribution in this device is somewhat more in doubt. The triaxial device may be criticized in that one stress component is applied uniformly through a yielding rubber membrane, whereas the other is transmitted through a rigid metallic testing head. The dissimilarity in the relative deformability of the specimen and testing heads and the restraint which the latter imposes locally on lateral expansion of the specimen may certainly be suspected of producing nonuniform stress distribution. The size and proportions of the specimens that may be tested in these devices are also a matter of interest. The specimens used in the triaxial tests are cylindrical in shape, the axial length varying from two to three times the diameter. The majority of the agencies use triaxial specimens less than 3 in. in diameter with the minimum diameter of 1 in. Several agencies, however, report using specimens up to 6 in. in diameter to obtain a more representative sample, and one contributor reports the construction of a device to test specimens 24 in. in diameter. Stabilometers are

generally exceptions to this rule in that the length of the test specimen is often equal to the diameter, a feature that may be criticized. The specimens are usually 4.5 in. in diameter. The direct-shear device takes prismatic specimens usually about 3 in. or 4 in. square by 0.5 in. or 0.75 in. thick, although several agencies report using specimens up to 12 in. square. The thinness of specimens tested in the direct-shear device is an advantage when it is desired to permit consolidation of the specimen during test, since the rate of consolidation varies as the square of the length of drainage path. The same characteristic of these devices, however, makes it impossible to make tests on soils that contain stones larger than $\frac{1}{4}$ in. in size. This fact alone has led some engineers to make use of triaxial testing equipment although they were not convinced that the latter was superior in any other respect. All of the double-shear devices use cylindrical samples. Most of these specimens are 3 in. long and 2.5 in. in diameter, although two agencies report using specimens from 1.5 in. to 4.5 in. in diameter.

Of the innumerable other subjects that might be discussed with reference to equipment, provision for volume change may be mentioned. It is generally believed that soil specimens during laboratory test tend to change volume even when loads are rapidly applied. If it is desired to maintain a constant load on a specimen in any given direction, the load must be applied through some device that is free to move with the specimen. In the double-shear equipment, even if the axial load is applied through movable heads, the specimen is likely to be restrained by arch action in the tube on either side of the center section. Thus, in all probability, the normal stress on the shear plane in this device does not remain constant; nor is the magnitude of the applied axial load a true measure of the normal stress. In modern direct-shear equipment, the normal load is generally applied by dead weight through a loading block and yoke which is free to move with the specimen, although some of the older direct-shear devices did not adequately allow for volume change of the sample. In respect to maintaining constant normal stress, the direct-shear device is considered superior to the double-shear device. However, the specimen in the direct-shear equipment is rigidly restrained by the box in a direction perpendicular to the plane of action of the normal and tangential forces. The stress in this direction must vary, therefore, if the sample tends to change volume. The triaxial test appears to be superior to both the others in this respect.

The foregoing applies to the testing devices. A word may be added about the equipment used to apply loads to these devices. The over-all impression gained from the illustrations furnished with replies to the survey and from viewing a number of different laboratories is that most of the existing loading equipment is distinctly amateurish in appearance and construction. In some instances, the collection of water cans, wires, bicycle wheels, hanging weights, levers, pulleys, and wooden framework looks as if it had been inspired by a well-known contemporary cartoonist instead of a competent engineer. It seems inevitable to the writer that makeshift equipment of this character cannot fail to create the impression in the minds of practical engineers that the test results are questionable, and in some instances it is true that test re-

sults have unknowingly been adversely affected by faulty operation of loading equipment. It is to be hoped that as testing methods become more standardized, the equipment will become more professional in appearance and construction.

With reference to testing procedure, the committee concludes that the determination of the Mohr rupture envelope, in whole or in part, is the common objective in all tests described by the contributing laboratories. There is a rather wide discrepancy, however, in the extent to which it is considered necessary to obtain this diagram in terms of effective or intergranular stresses rather than total externally applied stresses.

Two methods for controlling load application were reported: (1) Application of load so that rate-of-deformation or strain is controlled and (2) application of load so that rate-of-stress variation in the specimens is controlled. Testing procedure referred to as "constant strain" means that the test is so conducted that deformation of the specimen in the direction in which load is being varied occurs at a constant rate. A "constant stress" test is one in which the rate of stress increase produced in the specimen by external loading is maintained constant. One agency favors the use of a type of loading whereby the shearing stress is increased at a decreasing rate. In some laboratories the type of loading equipment used, or the prescribed testing method, is such that rate-of-strain variation or rate-of-stress variation must be accomplished by steps or increments rather than by steady variation of load.

Regarding the relative merits of controlling the rate of strain or stress, the former is considered to be better adapted to determining a complete stress-strain diagram showing ultimate as well as maximum shearing strength, than the latter. In the stress controlled tests, a sudden failure of the specimen may be produced at the maximum beyond which further control is quite difficult. For determining only the maximum strength, however, there seems to be little evidence that one test procedure is better than the other.

Further variation in testing procedure is found when different types of soils are considered. Nearly all agencies were in agreement in the testing of cohesionless soils. In general, volume change is permitted during shear, although for certain tests some agencies restrict volume change in a closed system test and measure the pore-water pressure, while others vary external stresses in such a way that no volume change takes place. These last-mentioned tests can only be performed in the triaxial shear apparatus.

For cohesive soils (which, incidentally, are almost always saturated wholly or in part), three methods of testing are used—the "quick," "preconsolidated quick," and the "slow" shear tests. In the quick test, the normal load or the minor principal stress (depending on type of equipment) is applied and the test started immediately at a relatively rapid rate so that the sample is not able to consolidate appreciably during or prior to test. In the preconsolidated quick test, the sample is allowed to consolidate fully under a normal or lateral pressure, before shearing forces are applied, after which the shearing process is conducted at a rapid rate so that no further consolidation or expansion can take place. A slow test, however, is one which is performed at such an ex-

tremely slow rate that the soil is completely adjusted to the shearing and normal stresses at all times. Most of the agencies reporting favored the use of preconsolidated quick tests. However, this was modified if the application to their particular problem dictated otherwise.

The basis reported by most contributors for selecting one of these three types of procedure appears to be a consideration of conditions which affect consolidation of the natural soil deposit. The survey indicated that there is a growing realization of the need for correlating shear-testing theories with the consolidation theory. For example, in the case of "undisturbed" cohesive materials, there is the necessity of restoring the specimen before testing as nearly as possible to the same conditions of void ratio, water content, and intergranular stress as existed in nature. There is then the need for conducting the test so that intergranular stresses are varied in the same manner as those in the prototype. It seems evident from even this brief discussion that shear-testing methods cannot readily be completely standardized.

Although the details of applying test data to design studies are infinitely varied, the committee finds that all methods reported have a common background in the general acceptance of Mohr's rupture theory, the test data being used in one way or another to indicate what combinations of tangential and normal stresses are critical for prototype conditions. Some of the reporting agencies follow the practice of establishing limiting positions of the rupture envelope for extreme conditions and then selecting some intermediate envelope suitable to a given problem by judgment. Some follow a conservative procedure throughout, without attempting to establish the upper limit of shearing strength. Most agencies consider that the rupture envelope is straight and that with suitable modifications it may be represented, at least over a limited range, by the Coulomb equation,

$$s = c + p \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which s = shearing strength; c = cohesion; p = normal, intergranular stress; and ϕ = angle of internal friction. Aside from considering $c = 0$ when dealing with coarse-grained soils, the principal modification is that, in cohesive materials, the value of the term $p \tan \phi$ is generally limited by using only the initially existing effective stress when loads are to be quickly applied in nature or only such increase in the initial effective stress as can be justified by consolidation of the deposit during lay-over periods in the construction schedule. The "preconsolidated" quick test is used quite often to furnish data for use in this method of application. One of the reporting state highway laboratories uses the shear test primarily to classify soils into types on the basis of the amount of strength or stability retained at various water contents. Two laboratories in private practice use the double-shear test primarily to determine safe bearing values for slab and pile footings. In both these cases, the test results are correlated to some extent with accumulated observations on prototype behavior.

One of the most vital factors to be considered in applying test data is the reliability of the analysis of the stresses that exist and those that are developed in the prototype. Except for a few references to use of the Swedish circle

method, practically no report was made by the contributing laboratories on this subject. The impression was gained from the survey, however, that many of the contributing agencies are more concerned with details of testing equipment and procedure than with development of adequate and dependable methods of stress analysis. There is even some reason to believe that less uncertainty is introduced in the final result through experimental errors in obtaining data than through serious misconceptions of stress conditions in the prototype. In this connection, reference might be made to a study sponsored in 1939 and 1940 by the Chief of Engineers, U. S. Engineer Department, in which analysis of a proposed structure was requested from a number of different agencies. The same test data were furnished to all, but due to differences in analytical methods a considerable variety of solutions was obtained, some indicating failure and some stability for the proposed structure.

Conclusions reached by the committee after review and consideration of the foregoing materials are as follows:

(*a*) No appraisal can be made of the value of any soils analysis by consideration of testing methods alone. Full consideration must be given not only to the type of testing equipment used and the test procedure followed, but also to the way in which the test data are applied.

(*b*) Of the three types of equipment reported as being in general use, the double-shear device seems better suited to empirical comparisons of different soil types than to determination of the actual shearing strength of soils. Choice between the direct-shear and the triaxial-shear testing equipment in any practical work depends on the nature of the individual problem. For research, the triaxial equipment appears to be more satisfactory. An exception to this is the stabilometer, a type of triaxial equipment with certain construction features which make it more suitable for empirical tests than for research.

(*c*) No testing equipment can be considered entirely satisfactory, except for empirical tests, until the state of stress that it produces in the specimen is definitely known. The survey did not indicate that any of the contributors had any knowledge of stress conditions in their various testing devices beyond that available from surmise or circumstantial evidence.

(*d*) It seems evident that testing procedure must be varied to fit each problem. Beyond classifying tests in general terms as "quick," "preconsolidated quick," and "slow" tests, the inherent nature of the work is such that no standardization appears possible or desirable. Correlation between shear and consolidation tests is considered important for cohesive soils. Realization of the fact that the ability of a soil to resist shearing stresses depends on the conditions of loading should be encouraged so that efforts to determine "the" shearing strength of a given soil will be discontinued.

(*e*) In spite of the important differences reported in the design of testing equipment and methods of test procedure and the inherent complexity of the problem, it is believed that the difficulties and importance of soil testing have been unduly magnified and that this has created the impression that soil testing is an end in itself. The committee believes, on the other hand, that in some cases in actual practice, the necessary data on soil characteristics may be

assumed by an experienced soils engineer without any tests at all, at least for preliminary design purposes, provided he has a clear conception of stress conditions in the prototype. The greatest weakness in current methods of soil analysis is believed to be in the determination of stresses in natural soil deposits before and after loading and in proposed earth structures during and subsequent to construction. In comparison with this weakness, dispute over details of testing procedure is considered as being of secondary importance. The survey reported herein did not indicate that completely reliable methods of stress analysis of soils are currently available; nor does the literature on this subject lead to this belief. It is concluded, therefore, that soil analyses dealing with the reaction of soils to shearing stresses must still be accepted with reservations and should be subjected to verification and review in the light of experience with existing structures.

ACKNOWLEDGMENT

For assistance in the preparation of this report, the committee is indebted to Harold A. Fidler, Jun. Am. Soc. C. E.

Respectfully submitted,

BENJAMIN K. HOUGH, JR., *Chairman*

October 16, 1941

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

GLENNON GILBOY,[1] ASSOC. M. AM. SOC. C. E.[1a]—The committee report is especially commendable for its clear outline of the technical problems which require further discussion and investigation. Without ignoring the importance of other items, the writer will confine the present discussion to one point on which the committee has laid particular stress—the relation between test results and practical application.

The committee points out that methods of design analysis have lagged far behind the developments in testing procedure and technique. This is the rule, of course, rather than the exception. It is too much to expect all branches of a subject to progress with absolute uniformity. Also, it would be unwise to delay a promising line of research merely because other lines are not as well advanced. Nevertheless, a practical method of design analysis, whether or not it is the ultimate solution to a problem, should at all times form within itself a consistent pattern; and the introduction of new techniques must always be accompanied by a re-examination of the problem as a whole to make certain that the pattern remains consistent.

[1] Cons. Engr., South Lincoln, Mass.

[1a] Received by the Secretary December 24, 1941.

The Swedish method of slope analysis, mentioned briefly in the report, will furnish a good example. The work of the Swedish Geotechnical Commission was finished in 1922.[2] It resulted in a direct and simple method of slope analysis that has been in general use ever since. It has been accepted not merely because of its directness and simplicity, but primarily because it was based on an exhaustive study of a number of actual slides, and was carefully correlated with the observed behavior of materials in Nature.

The correlation was based on the straight-line, or Coulomb, expression for shearing strength, with the constants determined by direct-shear tests. Since that time, considerable progress has been made, both in the technique of testing and in the basic concepts of shearing strength and associated phenomena. On the principle of using the best information available, it is only logical that the latest developments should be incorporated into the analysis. As soon as this is done, however, the basic pattern of the method is changed, and it is no longer correct to state that the Swedish field studies prove the practical applicability of the procedure. On the contrary, the revised method in that stage represents a purely theoretical concept, without any practical correlation. To obtain such a correlation from the original Swedish studies, it is necessary not only to repeat the analyses on the basis of the new concepts, but also to repeat all the soil tests on the basis of the new techniques. If this is not possible, there remain two alternatives—(1) either to accept the Swedish method in its entirety, outmoded though some of its concepts may be; or (2) to relegate it to the background in its entirety, and begin the accumulation of field data for a new correlation, using the new concepts and new techniques throughout.

This example illustrates the difficulties associated with any new development when it is applied to a practical problem. Much of the information previously accumulated is certain to lose its value, and the search for new data is sure to be laborious. Consequently, it is unwise, at the outset, to expect too much from new procedures and new techniques. No matter how significant an advance they may represent in the understanding of a particular phase, their practical value is necessarily limited by the rate of advancement of the entire problem to which they pertain.

A serious handicap to such advancement today is the almost complete lack of reliable measurements on the actual stress distribution in soil masses. To quote only one example, a large number of earth dams have been built in recent years with the aid of the latest developments in soil testing and stability analyses; yet it is safe to state that in practically no case has any one been able to formulate anything more than a very vague idea as to the correlation between the conditions assumed in the design and those actually existing in the structure. Even when failures have occurred, only the mass movements have been ascertainable—not the stress distribution; and in some cases even these have led to conflicting interpretations. For the want of reliable measuring apparatus, a vast storehouse of valuable information is locked tight, and will remain so until some ingenious soul finds the key.

It is hoped that the committee report, and the discussion arising from it, will aid in emphasizing the importance of this phase of the problem, and in stimulating future efforts to obtain the field data so sorely needed to complete the pattern.

[2] Statens Järnvagärs Geoteknisca Commission, Slutbetankande, May 31, 1922.

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DISCUSSIONS

REDUCTION OF MINERAL CONTENT IN WATER WITH ORGANIC ZEOLITES

Discussion

BY MESSRS. JAMES M. MONTGOMERY, AND GEORGE L. DAVENPORT, JR.

JAMES M. MONTGOMERY,[3] M. AM. SOC. C. E.[3a]—The results that are being obtained in water treatment make water-works chemists and engineers wonder what the plant of the future will be like. Mr. Goudey has presented an excellent review of the work that is being done in connection with the softening and demineralizing of water by means of base-exchange materials.

The discovery that certain synthetic resins have base-exchange properties is particularly promising for the future because of the infinite number of possible variations in the molecular structure of organic compounds, some of which will probably result in the production of improved materials. The basic principle having been established, organic research chemists no doubt will work to produce materials with greater exchange capacity, improved exchange efficiency, regeneration with less expensive materials, and the property of selective ion removal.

Water softening by base exchange, using either natural or synthetic siliceous zeolites, has been practiced for many years. As a result of this long, and in many cases sad, experience, the properties, both good and bad, of these materials are now well known and, where conditions are right, the materials are used with confidence.

On the other hand, the organic exchange materials are still in the development stage. It is true that carbonaceous zeolites have been used for several years and appear to give good results when used on either the sodium or hydrogen cycle, but the number of installations is too small and the time of service too short for the establishment of definite standards. It is known that the products of some manufacturers have shown a tendency to break down in service and produce black water. Synthetic resins are really in the development stage, and there are but few actual installations, some of which have been

NOTE.—This paper by R. F. Goudey, M. Am. Soc. C. E., appears on pp. 225–236 of this issue of *Proceedings*.

[3] (J. M. Montgomery & Co.), Los Angeles, Calif.

[3a] Received by the Secretary January 12, 1942.

disappointing to both customer and manufacturer. Probably a long period of experience, experiment, and education for both manufacturer and user lies ahead before all of the difficulties incident to the use of organic zeolites have been discovered and corrected. Such materials must have a useful life of several years, and unfortunately no satisfactory accelerated tests have ever been developed.

The properties possessed by organic zeolites are remarkable enough without some of the extravagant stories that have been told about them. Contrary to information furnished by certain popular science editors, it is impractical to use them to demineralize sea water. In fact, even if the materials were thoroughly proved by experience, the cost of demineralizing Colorado River water, as shown by Mr. Goudey, is too high to warrant consideration. High exchange capacity is claimed, but it has apparently been accomplished by sacrificing economy.

Messrs. Myers, Eastes, and Myers (1)[35b] have made determinations of the exchange values achieved by using quantities of salt ranging from 3.11 lb to 81.06 lb per cu ft of zeolite. The use of 3.11 lb per cu ft gave an exchange value of 8,400 grains per cu ft, and 81.06 lb per cu ft gave 29,800 grains per cu ft, with proportionate results for intermediate quantities. The exchange value of 8,400 grains was accomplished by using approximately twice the theoretical quantity of salt. This approaches the economy of good, synthetic, gel-type zeolites. The exchange value of 29,800 grains per cu ft required the use of approximately sixteen times the theoretical quantity of salt. In other words, it would cost eight times as much to operate at 29,800-grains-per-cu-ft exchange value as it would to operate at 8,400 grains per cu ft. Actually, the organic zeolites so far developed would seem to be approximately equivalent to a synthetic gel-type zeolite in both exchange capacity and regenerant consumption when operated in either the sodium, hydrogen, or anion removal cycles at economical exchange values.

When organic zeolites are used in the two-step process for removal of both anions and cations from a water such as the Colorado River supply, which contains a large amount of dissolved solids, the quantity of rinse water required to wash the excess of regenerating materials from the zeolite beds becomes a large item. Sulfuric acid, which is ordinarily used for regenerating the cation removal material, is considerably more difficult to rinse out of a zeolite bed than is salt, and both sodium carbonate and sodium hydroxide, which are usually used for regenerating the anion removal material, are even more difficult to rinse out than sulfuric acid. When the water softening plant of The Metropolitan Water District of Southern California was being designed, investigations were made of the two-step demineralizing process. Statistics furnished by the zeolite manufacturer at that time indicated that the quantity of rinse water that would have to be wasted would amount to 12½% of the total plant influent when treating average water and would increase to about 20% when treating the much harder water that is now stored in Lake Mead (Arizona-Nevada), Lake Havasu (Arizona-California), and Lake Mathews

[35b] Numerals in parentheses, thus: (1), refer to corresponding items in the Bibliography of the paper.

(California), as compared with 2% to 4% at the Colorado River Aqueduct softening plant.

If water could be demineralized economically, this would be an ideal process for reducing both the hardness and the total mineral content of Colorado River water. Some people fear that the high sodium percentage of the softened water will reduce its value for agricultural and horticultural purposes, although twelve years' use of softened municipal water at Beverly Hills, Calif., with a high sodium percentage, has not decreased the beauty of that city's gardens and estates. It seems probable, however, that the plan suggested by Oscar C. Magistad (8) for overcoming the effect, if any, of high sodium water would be satisfactory and practical.

In the Los Angeles (Calif.) area, more people seem to take pride in their lawns and gardens than in any other place in the United States. The beautiful landscaping seen on all sides is not produced as a result of planting and watering alone. Well-kept lawns are torn up, fertilized, and reseeded every few years, and flowers and shrubs require fertilizing frequently. If high sodium water should finally prove, after long use, to be unsatisfactory for domestic irrigation due to the removal of calcium from the soil by base exchange, it would seem that the application of calcium sulfate or possibly ground limestone along with the fertilizer would be both practical and economical. The cost would be small and probably would soon be accepted as a necessary expense incident to the maintenance of good lawns and gardens. The addition of a quantity sufficient to restore the proper calcium-sodium ratio should cost only a dollar or two per year for the average home.

Work such as Mr. Goudey is doing in investigating new base-exchange materials will help to speed the development of improved products. It is to be hoped that he will continue his researches and help to keep water-works chemists and engineers informed of future progress. The Metropolitan Water District, operating the largest zeolite softening plant in the world, also will continue its research, begun at Boulder City, Nev., in 1938. Facilities for such work at the new plant are excellent, and it is hoped that a great amount of constructive work will be done.

GEORGE L. DAVENPORT, JR.,[4] M. AM. SOC. C. E.[4a]—The writer's experience of ten years with the use of inorganic zeolite for treatment of water used in locomotives may be summed up as follows:

(1) It is very satisfactory for most waters of 15 grains per gal of hardness or less. The Atchison, Topeka and Sante Fe Railway Company does not usually treat water having less than 5 grains of hardness.

(2) When the hardness is much in excess of 15 grains, the zeolite process will produce treated water that will cause foaming and priming in locomotives, so the railroad must use a lime-soda treatment that will reduce the total solids.

(3) About 50% of the water treated with zeolite causes disintegration of the mineral in excess of the usual annual loss guarantee. Some softening

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[4a] Received by the Secretary January 8, 1942.

plants run as high as 14% loss of mineral per year. Even then the replacement cost is relatively small, and is insignificant as compared with the advantages gained.

(4) Corrosion of pipe lines and other steel surfaces coming in contact with the softened water can be avoided by using about 50% more wash water than is usually recommended by the manufacturer. It is the practice of the "Sante Fe" to determine the proper quantity of wash water by a silver nitrate test which guarantees that the plant will not be back in service until the salt content in the effluent is within 5 grains of the original salt content of the raw water. This is a very important point which, as a rule, zeolite plant manufacturers do not recognize, since they do not like to admit that so much water should be wasted.

(5) Although the "Sante Fe" has obtained estimates on organic zeolite plants, no convincing reason for their use in treatment of locomotive supplies has been presented by the manufacturers. In addition to their higher first cost, there is also to be considered the practical difficulty, in the case of hydrogen cycle plants, of handling acids as compared with salt solutions. Highly trained employees, such as are essential to a large municipal plant, are not available for operating railroad softeners. In fact, the tendency is to eliminate the human element by installing fully automatic zeolite plants, operated by electric motors, which only require occasional services of a man to keep the salt storage tank full.

(6) To summarize, the railroad softening plant is a valuable supplement to the lime-soda process, but cannot take its place for the treatment of most waters in the Western States. Some waters will cause much more loss of zeolite than manufacturers will admit is possible, but even in such cases the operating cost is reasonable and advantages far exceed disadvantages. Long life for piping and equipment using zeolite treated water can be predicted, if sufficient water is wasted in washing out the mineral during the regeneration process.

Use of inorganic zeolite does not appear practicable at the present for railroad water softening due to the high cost of apparatus and lack of skilled attendants. By spending extra money for fully automatic equipment, the ordinary zeolite plant can be made "foolproof" and will render outstanding service, if restricted to treatment of suitable waters.

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DISCUSSIONS

THE GREASE PROBLEM IN SEWAGE TREATMENT

Discussion

BY ARTHUR D. WESTON, M. AM. SOC. C. E.

ARTHUR D. WESTON,[4] M. AM. SOC. C. E.[4a]—The need for adequate means of grease removal in the partial treatment of sewage is well illustrated by the discharge into Boston Harbor and its tidal estuaries of some 244 mgd of sewage from a contributing population of about 1,840,000 persons in the Boston metropolitan area from three different outlets. The sewage from these outlets is chiefly of domestic origin and during the period 1935–1936 had a total suspended solids content from about 119 to 256 ppm, or 0.13 to 0.28 lb per capita per 24 hr; a 5-day B.O.D. from about 119 to 249 ppm, or 0.13 to 0.28 lb per capita per 24 hr; and ether-soluble fats from about 34 to 83 ppm, or 0.04 to 0.09 lb per capita per 24 hr. The diluting body of water into which this sewage is discharged is typical of the salt water of the harbors along the New England coast, is reasonably free from pollution except that from the sewers of the Metropolitan Sewerage districts and the Boston Main Drainage District, and has a chloride content of about 17,000 ppm. Oxygen tests of the harbor waters have shown ample oxygen. The sewage from the Boston Main Drainage District is discharged at the surface on the second and third hours of the outgoing tide from storage tanks, whereas the sewage from the North and South Metropolitan Sewerage districts is discharged continuously through outlets located from 30 to 50 ft below mean low tide. Definite nuisances have prevailed from bacterial pollution of the waters by the sewage and from slick or sleek fields.

Studies made under the direction of the writer[5] in 1935 and 1936 showed sleek areas as large as 2,940 acres in connection with the Boston Main Drainage System, 2,050 acres in connection with the North Metropolitan Sewerage System, and 6,820 acres in connection with the South Metropolitan Sewerage System. These areas are not only esthetically objectionable because of the use of the harbor waters for yachting and recreation, but the floating solids, con-

NOTE.—This paper by Almon L. Fales and Samuel A. Greeley, Members, Am. Soc. C. E., appears on pp. 193–209 of this issue of *Proceedings*.

[4] Director and Chf. Engr., Mass. Dept. of Public Health, Boston, Mass.

[4a] Received by the Secretary December 10, 1941.

[5] See "Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor and Its Tributaries," H. R. Doc. No. 1600 (Mass.), December, 1936.

sisting of wood, oil, grease, soap, and other debris, especially the grease and oil (which are characteristic of sewage and commonly found along the edges of the sleek areas), have been known to reach shore areas highly developed as summer resorts and bathing beaches, and areas normally well suited for the growing of shellfish. The authors cite Mr. Wright (3)[5a] as reporting grease balls, presumably from the outfall sewers of Boston, found at an isolated point 66 miles from Boston Harbor (see heading "Effects of Grease on Receiving Waterways").

With a view to remedying these conditions, numerous studies have been conducted to ascertain the most practicable means of treating and disposing of the sewage from the three sewerage systems. These studies included certain experiments to determine methods of removing grease, oil, and other matters from the sewage but with no thought in mind of salvaging the fats. Salvaging matters from sewage was investigated by the Boston Main Drainage District under Edgar S. Dorr and Robert Spurr Weston in 1911 (as mentioned by the authors under "Other Methods of Grease Removal"), and by the Massachusetts Department of Public Health under a resolve of the Massachusetts Legislature in 1917, particularly in regard to a patented process that involves[6] the addition of sulfuric or sulfurous acid to the sewage to precipitate the solids. H. W. Clark, at that time director of the Water and Sewage Laboratories and the Lawrence Experiment Station of the Massachusetts Department of Public Health, concluded that the recovery of grease from sewage was not economical and that the removal of organic matter by plain sedimentation was nearly as good as by the aforementioned patented process, and considerably cheaper.

The studies[5] made in 1935 and 1936 on the removal of fats from the sewage included small-scale experiments with the use of tanks on the sewage of the North and South Metropolitan Sewerage districts and the sewage of the Boston Main Drainage District, and large-scale experiments on the sewage of the Boston Main Drainage District. The latter experiments made use of one of the tanks of this district that is normally used to store the sewage for discharge on the second and third hours of the outgoing tide.

The small-scale experimental plant consisted of small tanks 2 ft by 5 ft in plan and 2 ft deep, equipped for fine screening, grease removal, and plain sedimentation. (The tank for screening had a channel 6 in. wide at the influent and effluent ends, which flared to a width of 18 in. at the center where screens of slotted yellow brass 18 in. by 21 in. could be placed.) The screens used in the experiments were of three sizes of slots: $\frac{1}{16}$ in., $\frac{3}{32}$ in., and $\frac{1}{8}$ in. by 2 in. long.

The small tank used for plain fill-and-draw sedimentation tests and for grease removal with aeration was 4 ft by 5 ft in plan and 5 ft deep. This tank was fitted on all sides except the influent side with sloping false bottoms 30 in. on 20 in. It had parallel wooden baffles spaced 1 ft 4 in. apart, dividing the tank into three channels and extending within a foot of the effluent end where a wooden cross baffle spread across the tank. All baffles extended to within about 6 in. of the bottom. The parallel longitudinal baffles had $\frac{1}{8}$-in. metal

[5a] Numerals in parentheses, thus: (3), refer to corresponding items in the Bibliography of the paper (see Appendix).

[6] "Report of Commission on Sewage Disposal, American Public Health Association," *Sewage Works Journal*, Vol. VII, No. 6, November, 1935.

baffles that could be adjusted to the level of sewage in the tank. The cross baffle at the effluent end extended above the sewage flow. When aeration was required, two removable porous diffuser plates were placed inside the center at the bottom of the center channel, and the sewage entering the tank at the end of the center compartment was forced over the longitudinal baffles and under and into the center channel again by a spiral motion. The grease collected on the surface was retained by the cross baffle. Air in the amount of about 0.1 cu ft per gal of sewage was applied.

The larger tank used for plain sedimentation, both for intermittent and continuous-flow experiments, was 4 ft by 7 ft in plan and 6 ft deep. Sewage entered this tank through two elbows that diverted the sewage back against the inlet end of the tank. About 1 ft from the influent end a cross baffle extended to within 1 ft of the tank bottom. A semicircular trough 6 in. in diameter, located about 1 ft from the effluent end of the tank and protected by a skimming baffle, was used to collect the settled sewage. Weir boxes 6 in. by 6 ft in plan and 6 in. deep were fitted with three baffles to smooth the flow at the entrance of the tank, and a triangular brass weir was used to measure the flow. The sewage flow during continuous-flow tests varied from 9 to 35 gal per min.

In the continuous-flow sedimentation tests, samples of the influent were collected as near as practicable to the influent pipe, and samples of the effluent were collected at the weir box. These samples were composite, consisting of $\frac{1}{4}$-pt portions collected every 0.5 hr over a period of 8 hr. The dissolved-oxygen tests during the continuous-flow experiments were on samples collected every 2 hr. The samples collected during the intermittent sedimentation tests consisted of composite samples of the influent collected during the period of filling of the tank and a 2-qt sample of the settled sewage collected at the end of the detention period. The dissolved-oxygen samples during the intermittent sedimentation tests were taken at the completion of the tank filling and at the

TABLE 7.—AVERAGE RAW SEWAGE ANALYSES
(Parts per Million)

Sewerage system	No. of samples	Suspended solids	B.O.D.	Kjeldahl nitrogen	Oxygen consumed	Albuminoid ammonia	Free ammonia	Chlorides	Fats
Boston Main Drainage:									
Calf Pasture, 1935.....	230	162	157	9.1	53	4.1	21.5	4,080	50
Moon Island, 1936.....	72	119	144	8.6	50	....	18.1	3,478	34
North Metropolitan:									
Deer Island, 1935.....	81	256	249	16.4	81	7.5	34.1	3,280	83
Deer Island, 1936.....	105	148	175	11.2	69	5.3	24.4	3,151	41
South Metropolitan:									
Nut Island, 1936.....	64	156	119	9.6	50	3.71	20.3	198	49

end of the detention period. During the pre-aeration sedimentation tests, composite samples of $\frac{1}{4}$-pt portions were collected every 0.5 hr over a period of 8 hr from the influent to the degreasing tank, the influent to the settling tank, and the final effluent. Dissolved-oxygen samples were taken at these locations every 2 hr. Table 7 shows the results of average analyses of the samples of

raw sewage collected during the tests from the three sewerage systems in question.

Table 8 shows the results of the analyses made during the plain sedimentation tests. This table includes only those samples having an initial suspended solids content of 100 to 300 ppm. The number of samples listed varied, in the

TABLE 8.—RESULTS OF PLAIN SEDIMENTATION TESTS OF RAW SEWAGE OF NORTH (N) AND SOUTH (S) METROPOLITAN SEWERAGE SYSTEMS

Description	One-Hour Detention			One-Hour Detention			Two-Hour Detention		
	N	B	S	N	B	S	N	B	S
No. of Samples[a]	14	19	10	16	9	6	15	9	10
Suspended Solids:									
Raw (ppm)	170	153	195	176	159	168	171	190	190
Effluent (ppm)	97	108	73	97	92	62	83	93	59
% reduction	42.9	29.4	62.6	44.9	42.2	63.1	51.4	50.9	68.9
B.O.D.:[a]									
Raw (ppm)	218	173	160	224	173	118	183	161	134
Effluent (ppm)	181	143	117	167	143	81	138	130	80
% reduction	17.0	17.3	26.9	25.4	17.3	31.4	24.6	19.3	40.3
Kjeldahl Nitrogen:									
Raw (ppm)	13.0	8.9	11.4	12.5	8.7	10.6	11.9	8.7	10.7
Effluent (ppm)	9.8	7.4	6.9	8.5	7.7	6.9	8.0	6.8	6.4
% reduction	24.6	16.9	39.5	32.0	11.5	34.9	32.8	21.9	40.2
Albuminoid Ammonia:									
Raw (ppm)	....	4.21	4.98	....	4.41	3.63	....	4.23	4.00
Effluent (ppm)	....	3.40	2.99	....	3.68	2.70	....	3.34	1.98
% reduction	....	19.3	40.0	....	16.5	25.6	....	21.0	50.5
Oxygen Consumed:									
Raw (ppm)	72	56	54	73	61	54	71	56	59
Effluent (ppm)	62	48	39	61	46	34	58	42	38
% reduction	13.9	14.3	27.8	16.5	24.6	37.0	18.3	25.0	35.6
Fats:									
Raw (ppm)	49	54	59	40	39	46	49	46	48
Effluent (ppm)	30	37	38	28	22	28	28	28	24
% reduction	38.8	31.5	35.6	30.0	43.6	39.1	42.9	39.1	50.0

[a] For exceptions to the number of samples, in the case of B.O.D. and fats, see text.

case of B.O.D. and fats, as follows (compare Table 8):

Detention time	B.O.D.	Fats
1 hr	17	18
1 hr	..	8
2 hr	7	..
3 hr	17	17
4 hr	10	..

The results of these small experimental plant tests showed that, although the fine screens removed the larger suspended and floating solids, they removed only a small amount of grease or fats, one of the principal causes of complaint of the discharge of sewage into Boston Harbor. Only 5% to 8% of the suspended solids were removed by fine screens as compared with about 50% by sedimentation. The authors mention these tests (see "Methods of Removing Grease from Sewage: Sedimentation and Skimming") and refer to the raw sewage of the Boston Main Drainage System where about 40% of the 46 ppm of grease could be removed by skimming and sedimentation (14). Pre-aeration

of the sewage was not successful in the over-all removal of fats and would indicate that the use of air alone, before sedimentation, in connection with these sewages would not be justified.

One of the existing storage tanks was used for the large-scale experiments on the sewage of the Boston Main Drainage District. This tank, which is

SEDIMENTATION EXPERIMENTS

age Districts and Boston Main Drainage District (B)

THREE-HOUR DETENTION			FOUR-HOUR DETENTION			FIVE-HOUR DETENTION			SIX-HOUR DETENTION		
N	B	S	N	B	S	N	B	S	N	B	S
16	18	9	16	12	7	10	11	2	10	9	5
187	156	150	166	136	136	192	172	121	147	173	145
80	77	55	66	70	51	58	82	28	55	65	58
57.2	50.6	63.3	60.3	48.5	62.5	69.9	52.3	76.9	62.6	62.5	60.0
250	192	126	198	186	115	178	203	84	185	174	114
181	142	76	148	136	73	120	139	42	139	115	71
27.6	26.1	39.7	25.3	26.9	36.5	32.6	31.5	50.0	24.9	33.9	37.7
13.3	9.6	9.2	12.5	8.7	9.1	11.5	9.3	6.0	12.1	9.0	8.3
8.8	6.6	5.8	8.2	6.2	6.0	7.4	6.9	3.8	8.2	5.3	5.8
33.8	31.3	37.0	34.4	28.8	34.1	25.6	25.8	36.7	32.2	41.2	30.1
....	4.40	3.64	....	4.28	3.27	....	4.47	2.15	....	4.27	2.74
....	3.14	2.28	....	2.78	2.04	....	3.18	1.00	....	2.66	2.18
....	28.6	37.4	....	35.0	37.6	....	26.6	53.5	....	37.7	20.4
73	58	51	75	53	47	66	59	41	76	56	43
58	47	33	58	46	32	49	48	28	61	42	31
20.6	19.0	35.3	22.7	13.2	31.9	25.8	18.7	31.7	19.8	25.0	27.9
58	52	57	36	45	51	41	57	25	42	62	51
36	29	30	17	28	31	21	34	15	21	26	36
38.0	44.2	47.4	52.8	37.8	39.2	48.8	40.3	40.0	50.0	58.0	29.4

trapezoidal in shape with sloping ends, is 162 ft wide and has an average length of 612 ft. In order that it might serve as well as practicable as a sedimentation basin, it was equipped with a dike and skimming baffle, and gates were installed to prevent sewage from flowing out at the bottom of the tank. Five of the outlets at the end of the tank were closed by bulkheads, and the two end openings were closed by sliding wooden sluice-gates. The dike was a triangular-shaped wooden structure about 9 ft high and 16 ft on the two equal sides, and extended symmetrically from the end wall of the basin to enclose one of the existing arched openings. The opening was the only outlet of the tank after bulkheading and gating the other openings, and it acted as an outlet for the tank during the experiments. The top of the dike formed the crest of two sharp-edged weirs each 13 ft long. The skimming baffle was a floating vertical wooden structure 3 ft deep, of which 2 ft were submerged, held 6 ft out from, and parallel to, the sides of the dike by spreaders. The spreaders were so connected as to permit the baffle to rise and fall with the level of the sewage in the tank.

In the first experiments the settled sewage was stored in the other three

remaining tanks used for the storage of the sewage for discharge on the second and third hours of the outgoing tide. In later experiments the settled sewage was permitted to discharge into the harbor continuously. Analyses were made of the raw and settled sewage during the experiments and, in the case of the continuous discharge, aerial photographs were taken to show the difference in extent and character of the sleek areas in the harbor resulting from the intermittent discharge of stored raw sewage, and the continuous discharge of settled sewage. Measurements made of the flow of sewage through the experimental tank showed that the average flow was about 60 mgd and that the detention period varied from 1.7 to 5.8 hr, with an average of 2.8 hr. Table 9 shows the results of these settling tests.

TABLE 9.—SETTLING TESTS, MOON ISLAND, BOSTON, MASS.
(Averages of 72 Samples)

Description	PARTS PER MILLION		% reduction
	Raw sewage	Settled sewage	
Free ammonia	18.1	18.8	....
Chlorides	3,478	3,437	....
Oxygen consumed	49.6	39.7	19.9
Kjeldahl nitrogen	8.58	5.94	30.8
Fats	34.2	20.0	41.5
B.O.D.	144	107	25.6
Suspended solids	119	51.5	56.7

It will be seen from Tables 7 to 9 that the raw sewage during these large-scale experiments was somewhat weaker than the sewage of the small-scale experiments. The samples collected from hour to hour showed that the sewage was strongest during the afternoon and early evening and weakest during the early morning hours. The purification obtained in the larger tank compared favorably with that of the smaller units.

It was found that the amount of scum, garbage, and other floating matter retained by the skimmers was much greater than expected, and rough estimates indicate that as much as 40 cu ft of such material per million gallons of sewage were held back.

Although he was not entirely satisfied with the results of the large-scale experiments (because the tank used was not well suited for sedimentation), the writer was convinced that a comparison of the results of the various experiments with the aerial photographs of sleek areas shows that proper settling and skimming of the sewage will result in improved conditions in the harbor by greatly diminishing the sleek areas.

Tests made during 1940 relative to clarification of the sewage at Nut Island included the use of certain coagulants. The raw sewage in this case had the following average composition:

Description	ppm
Albuminoid ammonia	3.73
Oxygen consumed	33
Turbidity	130
Suspended solids	115
Fats	27

The result of these experiments is given in Table 10. Tests made by stirring some of this sewage for 15 min, followed by 1 hr of settling, showed a 27%

decrease in turbidity over settling alone. Later tests were made to determine the effect of stirring the sewage prior to settling. In this case, since the sewage was received at different times, there was a decided difference in its strength, and each run was treated separately. Turbidity, suspended solids, and fats were determined for the raw sewage; for sewage that had settled 1 hr; for raw sewage that had been stirred 15 min and settled 1 hr; and for sewage that had been stirred 15 min with 8% by volume of sludge settled out of the five bottles, and settled 1 hr. Stirring the raw sewage gave an appreciable reduction in suspended solids over that due to settling alone. Stirring with additional sludge showed little effect. The use of the modified, rapid type of filter gave the best results (see Table 11, items 1 to 18). In the tests shown in Table 11, items 19 to 36, samples stirred with 10% (by volume) of returned sludge gave poorer results than stirring the raw sewage alone.

TABLE 10.—PERCENTAGE REDUCTION BY VARIOUS TREATMENTS OF SOME CONSTITUENTS IN RAW SEWAGE AT NUT ISLAND

Description	1-hr settling	4 grains[a] alum	5 grains[a] alum	10.8 grains[a] CaO	14.5 grains[a] CaO	12.6 grains[a] H_2SO_4	Modified rapid type filter
Turbidity	28	78	91	78	88	94	66
Suspended solids	55	75	88	..	..	..	83
Fats	47	80	..	..	..	..	90
Coli-Aerogenes	..	..	90	..	90	90	..

[a] Grains per gallon, plus sedimentation.

Tests were made on samples collected on February 27 and 28, 1940, of the raw sewage and of the sewage after 1 hr of settling. The sludge collected from five 1-gal bottles was settled and elutriated several times with tap water. About 10% by volume of this sludge was added to portions of the different sewages, aerated 10 min, and then stirred 5 min and settled 1 hr.

Another sludge was prepared by coagulating a gallon of sewage from Lawrence, Mass., with alum and collecting the precipitate. This precipitate was then mixed with another gallon of Lawrence sewage and the resulting sludge collected and concentrated; 10% of this sludge by volume was stirred 15 min with the different Boston sewages and then settled 1 hr. This last sludge was intended to show the effect of a return sludge from the precipitation of Boston sewage with alum. Lawrence sewage, however, was used in preparing this sludge because there was not enough sludge from the Boston sewage.

On the whole, stirring Boston sewage with added sludge has shown very little beneficial action over stirring the raw sewage alone, but with some lots of sewage the sludge did show improved results. It was conceded, however, that the action was so uncertain as to make its value doubtful, and there was a tendency for such sludge, after it had gathered up colloidal or fine suspended matter, to give it back later into the sewage being treated. This condition is similar to the condition found at the Lawrence Experiment Station from stirring ferric and manganese hydroxides in water. Table 11, items 37 to 54, shows the results of these experiments.

It was the opinion of George O. Adams, chief chemist at the Lawrence Experiment Station, that more satisfactory results could probably be obtained

TABLE 11.—EFFECT OF STIRRING PRIOR TO SETTLING

No.	Description	Raw sewage (ppm)	Percentage Reduction			
			Settling only[a]	Stirred[b]	Return sludge[c]	Rapid filter[d]
	9:00 a.m., February 7:					
1	Turbidity	150	43	61	46	75
2	Suspended solids	150	65	67	63	87
3	Fats	53	58	..	75	83
	11:00 a.m., February 7:					
4	Turbidity	100	37	56	42	72
5	Suspended solids	95	48	62	61	83
6	Fats	30	53	..	67	73
	1:00 p.m., February 7:					
7	Turbidity	135	53	64	63	77
8	Suspended solids	127	71	76	85	86
9	Fats	32	4	..	62	78
	3:00 p.m., February 7:					
10	Turbidity	200	35	55	65	74
11	Suspended solids	179	59	74	84	89
12	Fats	63	8	..	79	72
	4:15 p.m., February 7:					
13	Turbidity	150	38	56	65	77
14	Suspended solids	137	63	72	83	87
15	Fats	70	26	..	84	87
	Average of Five Samples (Items 1 to 15):					
16	Turbidity	147	41	58	56	75
17	Suspended solids	138	61	70	75	86
18	Fats	50	30	..	73	78
	10:00 a.m. to 1:00 p.m., February 13:					
19	Turbidity	130	49	59	42	71
20	Suspended solids	207	81	87	81	87
21	Fats	23	61	48	26	78
	2:00 p.m. to 5:00 p.m., February 13:					
22	Turbidity	130	43	56	48	66
23	Suspended solids	143	78	83	76	78
24	Fats	31	58	77	71	84
	6:00 p.m. to 9:00 p.m., February 13:					
25	Turbidity	260	45	58	57	64
26	Suspended solids	226	78	86	83	85
27	Fats	45	40	53	47	89
	10:00 p.m. to 4:00 a.m., February 14:					
28	Turbidity	205	37	51	54	59
29	Suspended solids	222	75	83	83	86
30	Fats	44	27	55	64	70
	6:00 a.m. to 9:00 a.m., February 14:					
31	Turbidity	155	45	55	55	62
32	Suspended solids	188	80	83	81	84
33	Fats	70	79	79	76	79
	Average of Five Samples (Items 19 to 33):					
34	Turbidity	176	43	56	52	64
35	Suspended solids	197	78	85	82	84
36	Fats	43	56	65	60	79
	9:00 a.m. to 12 Noon, February 27:					
37	Turbidity	105	38	60	45	14
38	Suspended solids	117	71	80	65	55
39	Fats	33	67	42	61	76
	1:00 p.m. to 4:00 p.m., February 27:					
40	Turbidity	150	40	60	59	57
41	Suspended solids	188	76	87	77	74
42	Fats	37	51	84	76	70
	5:00 p.m. to 8:00 p.m., February 27:					
43	Turbidity	270	35	84	56	44
44	Suspended solids	333	80	93	83	80
45	Fats	106	43	82	59	65
	9:00 p.m. to 3:00 a.m., February 28:					
46	Turbidity	300	38	52	67	65
47	Suspended solids	544	83	90	93	87
48	Fats	99	38	38	54	59
	5:00 a.m. to 8:00 a.m., February 28:					
49	Turbidity	225	37	54	64	59
50	Suspended solids	232	66	83	81	78
51	Fats	130	75	78	78	76
	Average of Five Samples (Items 37 to 51):					
52	Turbidity	210	38	62	58	48
53	Suspended solids	282	75	87	82	79
54	Fats	81	55	65	66	69

[a] Raw sewage, settled 1 hr. [b] Stirred 15 min and settled 1 hr. [c] Stirred with return sludge and settled 1 hr.
[d] Raw sewage passed through modified, rapid type of filter.

from these experiments if the period of stirring were longer, preferably twice as long. It also is his opinion that the reason for greater removal by plain sedimentation of the sewage from the South Metropolitan Sewerage System than is found in some other places is due to the presence of salt water in this sewage, resulting in the precipitation of the soluble soap as insoluble calcium soaps, thereby increasing the fats removed by plain sedimentation. He was of the opinion that 20% more fat could be removed by aeration than by stirring, these fats to be removed chiefly in surface scum which could be skimmed off. It was interesting to note that in the plain sedimentation tests there was an average removal of 71% of the suspended solids in 15 min, whereas 15 min of stirring increased the removal to 81%. It was also his opinion that aero-chlorination would be desirable, provided that the results warranted the cost; but it was his opinion that aero-chlorination would have little effect on the subsequent removal of the suspended solids.

Experiments were made in 1934 by the Massachusetts Department of Public Health in connection with the disposal of sewage from the South Essex Sewerage District for the removal of fats. The sewage of this district is from Salem and Peabody, Mass., where there are many tanneries and other industries having wastes containing considerable quantities of fats; from the City of Beverly, Mass., where the sewage is chiefly domestic in character; and from a large mental health hospital and certain other institutions. The average flow of sewage from this district at the time the tests were made was about 13 mgd (with average flows during weeks of minimum flow of 7.27 mgd and average flows during weeks of maximum flow of 25.80 mgd). It is estimated that the total population served at that time was 94,850. The sewage disposal works of the district are such that the sewage from Beverly and the institutions is pumped into the same outfall sewer that receives the sewage from Salem and Peabody. The point of outfall is in the outer limits of the Salem-Beverly harbors, about 8,300 ft from shore, where the water at low tide is about 30 ft deep.

The Salem and Peabody sewage combined had a suspended solids content equivalent to about 1.33 lb per capita per day and a fat content of about 1.15 lb per capita per day. The combined sewage from the City of Beverly and the institutions had a suspended solids content of 0.14 lb, and a fat content of 0.05 lb, per capita per day.

The experiments for the removal of fats from the sewage were in connection only with the sewage from Salem and Peabody and consisted of tests to determine the amount of fats removed by fill-and-draw sedimentation, by aeration followed by sedimentation, and by continuous flow with the use of a skimming baffle. Two circular wooden tanks of 2,000-gal and 400-gal capacity were used, into which the sewage was pumped from the trunk sewer of the system at a point about 1 ft below the normal flow line in the sewer. The connecting piping was so arranged that the tanks could be operated in series or in parallel. When air was applied, the average rate of diffusion was 0.18 cu ft per gal of sewage through a filtros plate set in the bottom of the tank.

The fill-and-draw, plain, sedimentation tests without the use of air showed

that, with a detention period of 7 to 10 min, the removal of fats was 57.9%, or about 400 lb per million gallons of sewage treated. The amount of fats removed by skimming after 7 min of sedimentation was equal to 0.6% of the total fats in the sewage treated. Tests made with the application of air for a period of 7 min, followed by quiescent sedimentation for an additional 7 min, showed a removal of fats of 57.3%, or 425 lb per million gallons, and the fats removed by skimming were equal to about 2.0% of the total fats in the raw sewage. It should be stated that the tests with and without the use of air are not wholly comparable as the sewage to which the air was applied had a higher fat content than the sewage which received no air application.

In the continuous-flow experiments, a detention period of 37 min was provided by throttling the pump delivering sewage to the tank. Here the fat content of the sewage was 108 ppm as compared with 83 ppm in the fill-and-draw tests and 89 ppm in the test with air. This continuous-flow test showed a

TABLE 12.—SUMMARIZED RESULTS OF EXPERIMENTS
(Parts per Million)

Treatment	No. of runs	Detention period (min)	Fats				Fats skimmed (% of total)	Suspended Solids				Total Chlorine[a]		Air applied (cu ft per gal)
			ppm		Removed			ppm		Removed				
			Raw	Effluent	%	Lb per million gal		Raw	Effluent	%	Lb per million gal	Raw	Effluent	
(a) AVERAGES														
Sedimentation[b]	23	7	83	35	57.9	400	0.6	569[c]	238	58.2	2,760	...	...	....
Aeration[d]	18	7	89	38	57.3	425	2.0	..	..	...	...	...	...	0.175
Skimming[e]	6[f]	37	108	49	54.4	484	...	633	293	53.7	2,840	...	...	....
(b) MISCELLANEOUS RUNS														
Sedimentation[b]	1	10	124	55	55.7	575	...	645	235	63.7	3,420	...	...	....
Plus salt[g]	1	7	46	18	60.9	233	2.2	..	..	...	...	3,600	7,500	....
Plus chlorine[h]	2	7	90	34	62.2	466	0.9	..	..	...	...	...	...	....
Aeration[i]	1	7	30	4	86.7	217	2.6	..	..	...	...	...	...	0.22
Plus chlorine[j]	2	7	91	35	61.6	466	2.4	..	..	...	...	...	...	0.18
Plus salt[k]	1	7	46	12	74.0	283	4.0	..	..	...	...	3,600	6,800	0.18

[a] Chlorides. [b] Fill-and-draw sedimentation. [c] Determined in only two runs. [d] Aeration followed by sedimentation. [e] Continuous flow, with skimming baffle. [f] Excluding certain preliminary runs. [g] Sedimentation with 6.75 lb (per 100 gal of sewage) of common salt added. [h] Sedimentation with 10 ppm of chlorine. [i] Sedimentation before and after aeration, with 46.5 ppm of chlorine added. [j] Sedimentation after aeration, with 10 ppm of chlorine added. [k] Sedimentation before and after aeration, with 6.75 lb of salt per 100 gal of sewage added.

removal of fats of 54%, which is equivalent to 484 lb of fats per million gallons of sewage.

In these experiments an average of ten samples of the sludge that settled out in the tanks during various forms of sedimentation contained 16.2% of fats. Tests also were made in connection with the sewage of Salem and Beverly to determine the separation of fats from the sewage by the use of salt and chlorine.

In the salt tests, sodium chloride was added in amounts of 6.75 lb per 100 gal

of sewage. The sewage in this case had a fat content of only 46 ppm. After thorough mixing, followed by a quiescent period of 7 min, the effluent had a fat content of 18 ppm, showing a fat removal of 60.9%, of which 2.2% was represented in the skimmings. Using the raw sewage with a fat content of 46 ppm, adding salt at the same rate, and aerating for a period of 7 min with 0.18 cu ft of air per gallon of sewage, followed with a quiescent period of 7 min, the fat content was reduced to 12 ppm, a reduction of 74%, of which 4.0% was represented in the skimmings.

Chlorine in amounts of 10 ppm applied to sewage containing 90 ppm of fats, followed by a quiescent period of 7 min, reduced the fats in the effluent to 34 ppm, a reduction of 62.2%, of which 0.9% only was in the skimmings. Chlorine in amounts of 10 ppm applied to a sewage containing 91 ppm of fats, followed by aeration for 7 min at the rate of 0.18 cu ft of air per gallon of sewage and a quiescent period of 7 min, reduced the fat content to 35 ppm, a reduction of 61.6%, of which 2.4% was in the skimmings. The addition of 46.5 ppm of chlorine to a sewage having a fat content of 30 ppm, followed by aeration for 7 min at the rate of 0.18 cu ft of air per gallon of sewage and a quiescent period of 7 min, resulted in a fat removal of 86.7%, of which 2.6% was in the skimmings.

The results of the analyses of the samples collected during these experiments are shown in Table 12 and confirm the results of the experiments made on sewage discharged into Boston Harbor, in that the additional amount of removal of fats by aeration did not appear to warrant the addition of aeration to plain sedimentation of the sewage without further treatment for the removal of fats.

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DISCUSSIONS

SURFACE RUNOFF DETERMINATION FROM RAINFALL WITHOUT USING COEFFICIENTS

Discussion

BY ALBERT S. FRY, M. AM. SOC. C. E.

ALBERT S. FRY,[37] M. AM. SOC. C. E.[37a]—The authors have made a valuable contribution to a field of hydraulic engineering which, in many respects, is in a pioneer phase of development, with arguments pro and con as to the practical usefulness of infiltration theory and procedure. Progress in this field will be made as basic knowledge accumulates and as engineers become better acquainted with infiltration and its advantages and limitations.

It is to be regretted that the paper deprecates past work in hydrology and glorifies infiltration as a new method that puts hydraulic design on a parity with structural steel design in exactness. It is true that the network of stream gages and rain gages has expanded in certain areas of the United States in recent years, providing valuable information of particular use in those areas. The authors expect this to revolutionize engineering practice but overlook the fact that much of the new data are of limited local application and that overenthusiasm in some places has produced data of dubious value. To imply (as the paper does) that, in the past, design of important hydraulic structures has been "based on crude over-all general relationships in the hydrologic field," shows a lack of appreciation and recognition of the invaluable contributions made by men such as C. S. Jarvis, M. Am. Soc. C. E., and others working in this field. Hydrologic data—both past and present—are seldom available on any project to the extent that the engineer desires, but this has not prevented him from understanding hydrologic fundamentals. He has not only understood them but has recognized their limitations in practical application probably better than the disciples of the new order of

NOTE.—This paper by W. W. Horner, M. Am. Soc. C. E., and S. W. Jens, Assoc. M. Am. Soc. C. E., was published in April, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1941, by L. L. Harrold, Assoc. M. Am. Soc. C. E.; October, 1941, by Messrs. C. E. Ramser, LeRoy K. Sherman, A. J. Schafmayer, C. S. Jarvis, G. W. Musgrave, and F. L. Flynt; and December, 1941, by W. I. Hicks, Esq.

[37] Head Engr., Hydr. Data Div., TVA, Knoxville, Tenn.

[37a] Received by the Secretary December 19, 1941.

infiltrationists. Real progress is not achieved in any field by exorbitant and unwarranted claims.

The paper presents the method for surface runoff determination as it relates specifically to urban storm drainage but also recommends it as generally applicable to all drainage basins, and states that it provides a more logical and dependable basis of design for important hydraulic structures than has heretofore been considered feasible. This is more likely to be true of urban storm drainage than of large-scale uni-purpose or multi-purpose projects. The former deals with relatively small areas of physical characteristics susceptible of fairly close determination under detailed study, and design is predicated upon relatively short-time intense rainfall. The latter embraces large watersheds of complex physical and hydrological characteristics difficult of determination, and design is based upon rainfall over a longer period of time.

These large water projects involve extensive works such as reservoirs, channel improvements, and levees, and it is doubted that a more logical and dependable design for these works can be made by substituting infiltration for previously used methods. Such projects must be designed for maximum flood conditions, allowing a factor of safety that might erroneously be considered to be a factor of ignorance, whereas actually too much engineering judgment and experience are involved to apply the term "ignorance." During storm periods that cause maximum floods, the rate of runoff is high—in many cases 90% or more of the rainfall. Infiltration is a relatively minor part of the total rainfall during such great storms and hence is not a primary factor in determining the design basis for a uni-purpose project such as flood control or a multi-purpose project involving flood control, hydroelectric power, navigation, and other purposes.

There is as yet no sound method of estimating accurately the size of the maximum possible storm. Attempts to do this have been made in recent years and represent a forward step in the right direction. However, meteorological scientists, such as J. Bjerknes and C. G. Rossby, do not consider that the science has yet advanced to the point where the factors which produce maximum storms can be closely evaluated so that the maximum storm for a given region can be determined with any degree of reliability. If it were possible to know the maximum storm for which to design, then there would be more justification for the application of infiltration methods. Even in that case, for large areas approaching or exceeding 1,000 sq miles, for proper application of infiltration methods detailed information would be needed for the entire watershed regarding hydrologic factors, physiography, geology, and land use and management, including both agriculture and forestry. Approximations and estimates would have to be made at various steps, and the end result would not be any more sound than it would be by the methods heretofore used by competent hydraulists.

The authors' statement (see heading "Application to Hydraulic Engineering: III—Adjustment of Infiltration-Capacity Values to Antecedent Conditions and Precipitation Pattern") that infiltration procedure allows "the use of much lower factors of safety than one involving a choice of a coefficient of runoff" is not in accord with established facts. Until the engineer can

know with certainty the maximum storm that may be expected, there is no basis for expecting infiltration analysis to justify lower safety factors.

The files of the Miami Valley Conservancy District, and its technical publications, will show that the hydraulic engineers, headed by Sherman Woodward, M. Am. Soc. C. E., in charge of the design of the noteworthy flood-control reservoirs and channel improvements in the Miami Valley in Ohio were fully aware of infiltration and other hydrologic factors that are woven into the fabric of the infiltration theory advanced during the past few years. The paper refers to the Miami District research on small plots. Such experiments were made and reported upon by Ivan E. Houk, M. Am. Soc. C. E. They served to evaluate the characteristics of the particular plots, and were interesting within themselves, but furnished no basic information used in the project design. In that District, as in any other large water-control project, the design must provide for storms the size of which, in the final analysis, must be based on the best engineering judgment of a competent engineering staff, supplemented by qualified consultants. Such a determination is a practical integration of hydrologic relationships based on mature consideration of all factors involved.

In the design of flood-control works in the Tennessee Valley to provide protection to the City of Chattanooga, and in determining flood-control capacity in the Kentucky Reservoir at the mouth of the Tennessee River as a link in the flood-protection works for the Mississippi Valley below Cairo, Ill., a board of consulting engineers was called in to work with the Authority's engineers in determining the basis for design. This Board consisted of Daniel W. Mead, Past-President and Hon. M. Am. Soc. C. E., the late Harrison P. Eddy, Past-President, Am. Soc. C. E., and Gerard H. Matthes and Ivan E. Houk, Members, Am. Soc. C. E. The judgment of this Board (plus that of the Authority's own engineers, including Joseph H. Kimball, James S. Bowman, and Mr. Woodward, Members, Am. Soc. C. E.) developed design criteria supported by long and successful engineering experience and practice. It is scarcely conceivable that any infiltration derivative could be substituted for the considered decisions of such an engineering group.

The Tennessee Valley Authority has under preparation a plan for flood control for the French Broad River Basin in western North Carolina, an area of about 1,800 sq miles, including the City of Asheville, industrial properties of great value, and a rich agricultural area. A study is being made of the effects of improved land use and better farming methods on flood flows and water conservation in this region. Since 1935 the Authority has conducted an agricultural test program in this watershed involving more than 100 test demonstration farms. It has been clearly demonstrated that, by better land management and the proper application of lime and phosphate fertilizer, definite improvement in infiltration rates and capacity has been brought about that can be evaluated quantitatively in its effect on flood peaks and volumes. However, it is extremely difficult to make practical use of this fact in flood-control design, although supported by concrete information, because the engineers themselves cannot say with positive knowledge just how big a storm should be the basis for flood-control design. Whatever the basis finally

determined upon, it must be the best judgment of the engineers and must be large enough to be amply safe to protect the valuable city and industrial properties involved. It would be foolish to apply a method to the extreme flood—the design flood—which might result in lowering the peak a matter of inches when the size of that flood must be estimated with the probability of considerably more error than the total beneficial effect of improved cover under good land management.

The authors recommend aerial photographs for land-use information to determine percentages of land within a watershed having particular types of vegetal cover, such as forests, pastures, and cultivated land. The Tennessee Valley Authority has made extensive use of aerial photographs in its mapping and other programs and has available excellent aerial photographs of the entire 41,000 sq miles in the Tennessee Valley. It is possible from these photographs to determine the percentage of open and forested lands, but for infiltration purposes much more than this is necessary. However, there are great differences in lands with respect to infiltration capacities that cannot be discerned from aerial pictures no matter how good these may be. For example, it is impossible to determine, from the aerial photographs, the extent to which a forest canopy is complete or the amount and character of litter under the trees. Woodlands used for pasture have quite different infiltration characteristics than woodlands not so used, yet both would appear the same in an aerial photograph. In studying the effects of land use on flood control in the French Broad watershed by infiltration methods, it has been necessary for foresters and agriculturalists to go into the field and classify the various types of land use into a number of different classifications for each major usage. Even this work has had to be generalized because of the large watershed involved.

The paper also states that soil maps are available for a large part of the United States. This is literally true, but it fails to tell the real status of the soil surveys. These are of three general types. There are the reconnaissance maps, covering large areas, which are so generalized that they do not provide the detailed information desirable for infiltration procedure. Detail surveys by counties are available for only 35% of the land area of the United States. There are the much improved detail surveys issued only in the last few years for a small number of counties. The last two types are valuable in providing useful information for infiltration studies, but these are not available for about two thirds of the country.

The unwary and unknowing would be led to believe that with aerial photographs and soil maps it is a simple matter to classify the land-use and soil characteristics of a drainage basin sufficiently for a satisfactory application of infiltration. Experience in application belies this impression. It is this kind of statement that leads thinking engineers to discount infiltration as a useful engineering tool, and it is unfortunate that the infiltrationists have adopted this technique.

From this discussion so far, it may appear that the writer is an "anti-infiltrationist" but, as a matter of fact, he is enthusiastic about the possibilities of this method when used in its proper place. Infiltration has a definite

and most useful application in a field not mentioned by Messrs. Horner and Jens. This field is exemplified by the water-control operations of the large system of multi-purpose reservoirs in the Tennessee Valley. The system now (1942) numbers sixteen reservoirs and nine more are under construction so that the dispatch of water through these reservoirs, operated as one integrated system, is quite complex. A group of especially trained hydraulic engineers make detailed determinations of stream flow each day, including both surface and subsurface runoff, from ungaged areas. The efficient operation of the system requires continuous close knowledge of water available throughout the Tennessee Valley in numerous headwater streams and in local areas tributary to the reservoirs. In this work of daily river-flow predictions and water-control operations, infiltration has large possibilities. The condition of sub-basins with respect to physical and geological characteristics, ground-water levels, field moisture, cover, and land use can be known in advance of rain so that, when the rain comes, these facts can be considered and infiltration methods used to estimate storm runoff. The ordinary storm is not the great storm for which the dams and other works are safely designed, but rather the one that is likely to occur at any time, depending upon the season of the year. In this class of rains, where infiltration may account for a considerable part of the rainfall, the method is of real importance and affords a logical approach to the problem of estimating runoff.

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DISCUSSIONS

COMPACTION OF COHESIONLESS FOUNDATION SOILS BY EXPLOSIVES

Discussion

BY PAUL BAUMANN, M. AM. SOC. C. E.

PAUL BAUMANN,[7] M. AM. SOC. C. E.[7a]—Presented with military brevity and precision, this paper is very timely indeed and should prove to be of distinct significance in view of many floating dams that have been built or are proposed to be built in connection with the national flood control program.

The consolidation of granular foundation materials by means of shock or vibration, as a result of blasting, stands in contradistinction to the chemical methods of solidification of similar materials. In principle, the first method consists of rocking the particles into closer contact, whereas in the second case the particles remain undisturbed and the voids between them are filled partly or completely by a suitable combination of liquid chemicals. Both methods have at least one feature in common—namely, the difficulty of access and inspection as well as sampling to ascertain the results of the treatment. Hence, the control of the stabilization of foundations of this character is definitely at variance with the control of the stabilization by means of compaction of embankments above the ground surface. The latter are ordinarily built in relatively thin layers, offering ready access to testing at all times.

The blasting technique, as described by the author, resembles certain methods of demolishing mine tunnels or underground communications of various types well known in military engineering, particularly in connection with field fortifications. One of the many tasks of the sapper is the demolition of such enemy underground passages without appreciable disturbance of the ground surface—that is, especially without the formation of a crater or a "mole hill." This is accomplished by the so-called squash mines that have a striking resemblance to the type of mines described in the paper.

Contrary to the normal crater charge with a sphere of influence touching the ground surface as shown in Fig. 14 by the solid circle, the sphere of influence of the squash mine is much smaller, possibly as indicated by the dotted circle. Expressed in kilograms and meters for the weight W of the dynamite charge

NOTE.—This paper by A. K. B. Lyman, M. Am. Soc. C. E., was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1941, by Reuben M. Haines, Assoc. M. Am. Soc. C. E.; and January, 1942, by Messrs. R. G. Hennes, and John D. Watson.

[7] Senior Asst. Chf. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

[7a] Received by the Secretary January 3, 1942.

and the radius R of the sphere of influence, respectively, the simple relation

$$W = c \times R^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

exists, in which c is a coefficient that approaches unity for a 60% dynamite placed in fully tamped blast holes in medium-fine sand.

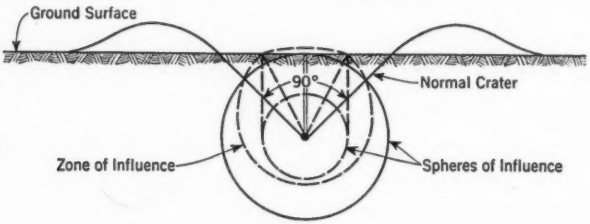

FIG. 14

According to the author's information, best results were obtained, at least in one location, if a charge of 8 lb of dynamite was used 15 ft below the ground surface. For the normal crater charge that would result in a 90° cone, more or less, the formula would call for approximately 200 lb of dynamite. Hence, the charge actually used was one twenty-fifth of the normal crater charge. Ordinary squash mines would probably range between one tenth and one fifteenth of the normal crater charge, depending on the character of works subject to demolition. Therefore, they would receive roughly double the charge as used for best compaction. This means that test pits within the blast area, if adequately timbered, would not be subject to destruction from compaction blasts unless they were located within the zone of influence—that is, in this case, within about 5 ft of the charge. Hence, there appears to be no reason why, with a relatively small additional expense, a more rigid control of the process could not be attained.

It is to be expected that compaction by blasting would result in somewhat erratic degrees of compaction. This is particularly notable in the data shown for the Denison Dam where the foundation material within the range of blasting was not saturated. A saturated condition is undoubtedly very desirable for more uniform propagation of the pressure waves, as the author clearly states. In studying Fig. 11, and particularly the part applying to the North River test area, it is significant that, at about 13 ft of depth, the density of the material was greater before blasting and that below some 18 ft of depth the trend of the curve showing the conditions after blasting is in the direction of diminishing density with increasing depth. In this connection, the author's emphasis of the need for further investigation before definite conclusions can be drawn from results so far obtained is believed to be well supported.

The writer had occasion to make studies and to direct tests on the solidification or petrifaction of granular material by means of chemical methods. Although this was a number of years ago and prices of chemicals may have changed considerably in the meantime, it is the writer's conviction that any one of the feasible chemical treatments to accomplish what apparently was accomplished by means of blasting would have been far more expensive than the latter method.

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DISCUSSIONS

COST OF PUBLIC SERVICES IN RESIDENTIAL AREAS

Discussion

BY T. T. MCCROSKY, M. AM. SOC. C. E.

T. T. MCCROSKY,[11] M. AM. SOC. C. E.[11a]—Many large-scale real-estate developers have made detailed cost studies of the relative economics of private building operations on high-priced improved slum land versus low-priced raw land; but their studies do not include consideration of the costs to the public that enter into their contemplated developments. To the writer's knowledge, Mr. McHugh's paper is the pioneer research in the field of cost of public services made necessary by large private residential projects. As a former colleague of the author, it was the writer's privilege to follow this study closely during the months that it was being conducted without, however, participating directly in its progress.

Similarly thorough analyses could be made for other cities, with different unit costs and appropriate modifications in the assumptions; but the resulting quantitative comparisons would assuredly be of the same general order of magnitude, differing only in degree. The case is proved by this study. It costs the public very much more to provide services for outlying developments than for equivalent redevelopment projects in "near-in" locations, where many satisfactory public facilities already exist. Whether it be 50% to 90% more per person housed, as computed by the writer (for Units A_1 and A_2, where rapid transit charges are excluded), or some moderately lesser or greater figure, is of secondary importance to the fundamental conclusion that it is manifestly uneconomical from the public standpoint to encourage the decentralization of cities.

The great difference in public costs between development of areas with existing transit facilities and areas where new transit lines must be provided is clearly revealed by the author. The moral is inescapable that the least a

NOTE.—This paper by F. Dodd McHugh, Esq., was published in June, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1941, by Donald M. Baker, M. Am. Soc. C. E.; and November, 1941, by Howard Whipple Green, M. Am. Soc. C. E.

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[11a] Received by the Secretary January 16, 1942.

city should do is to discourage new large-scale building on outlying vacant land that does not have, but will require, transit service.

It would be very valuable to have an extension of this study, in order to develop consolidated comparisons of capital and operating figures for the combination of public services plus private project costs. The differentials of public outlay are both capital and current account. They result from the cost of duplicating facilities and services already existing at the central location. These have been very thoroughly and ably analyzed by the author. The principal differentials of private outlay are capital items. They are the price of land, the expense of assembling it, the cost of the improvements, and the cost of the demolition of slum buildings to make way for the new project. Unfortunately, slum buildings do not tumble down like the walls of Jericho at the summons of a Joshua.

With a consolidated cost comparison, it would then be fruitful to study the feasibility of offering some partial public subsidy to the private builder to encourage him to redevelop slum areas. This could be scaled so that the costs to the taxpaying public, including subsidy, still would be well below what they would aggregate if the builder developed outlying raw acreage. In practice, the subsidy might often take the form of giving title deeds to tax-foreclosed properties, which were foreclosed because they yielded no taxes and thus made no return to the public, and which can never make a return unless transferred to a private owner for sound redevelopment. Another form of subsidy that costs the public nothing is to deed unnecessary street land to building corporations. Partial or limited tax exemption is also "cheap at the price," inasmuch as slum land is notoriously delinquent in most cities. If the public actually collects moderately more taxes from the new project than it did from the substandard properties that are replaced, it is "ahead of the game."

When peace comes, urban America will need a coordinated program for city rebuilding, embracing all price and rental brackets and clearly determining the extent and limits of public and private participation in each bracket. Preparation of this program is the responsibility of the technical professions in the building field. Mr. McHugh's paper is a valuable contribution to the economics of this broad-scale approach to urban redevelopment.

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DISCUSSIONS

DEVELOPMENT OF TRANSPORTATION IN THE UNITED STATES

Discussion

BY MESSRS. FRED LAVIS, AND WILLIAM J. WILGUS

FRED LAVIS,[6] M. AM. SOC. C. E.[6a]—The author merits the thanks of all those interested in problems of transportation for having presented such a comprehensive and statistically well-fortified review of transportation developments during the last century, as well as a broad picture of the internal transportation system of the United States as it now exists.

Too many people have only a narrow and meager comprehension of the entire problem which, as the author states in the "Synopsis," is one "so vital and so far-reaching that a sound national economy may depend on its solution."

The author has painted a fair and unbiased picture of the entire situation. He only alludes very briefly, however, to the vast number and great importance of the many indeterminate factors that prevent any reasonably accurate determination of relative costs of transportation by most of the facilities in use today.

What is the value of the property used by the railways? How can such value be determined? What are the costs of highways—original cost, reconstruction, and maintenance? How are the costs of providing and maintaining highways to be distributed among, through long-distance traffic, local freight traffic, and ordinary automobiles used for business, for pleasure, and for access to property?

What are the effects of curvature, varying rates of gradient, rise and fall, distance, smoothness and width of pavement, separation of lanes of traffic, separation of cross traffic, etc? Should drivers of motor vehicles who benefit by improvements pay a special tax for them?

What part of highway costs may be chargeable to military use?

How do railways or highways or even waterways affect the values of adjacent, or more or less near-by, properties? To whom do accretions of value belong? For instance, on a highway like Route 25 in New Jersey through

NOTE.—This paper by J. E. Teal, M. Am. Soc. C. E., was published in December, 1941, *Proceedings*.

[6] Cons. Engr., New York, N. Y.

[6a] Received by the Secretary December 30, 1941.

Jersey City, Newark, and Elizabeth, how much of its cost or value should be allocated to traffic between the remainder of the United States and New York, and how much to the State of New Jersey? How much are the cities through which it passes benefited by having this traffic taken off their streets?

What proportion of the costs of the Westchester parkways should be borne by the county, the state, and the City of New York? Should the traffic which uses these parkways, and thereby saves time, tires, and gasoline, pay a proportionate part of their cost? A large book could be written about these and other questions and difficulties of determining costs of transportation.

The associations of highway builders and users have proved to their own satisfaction that all highway costs are covered by taxes paid by their users. The railways have proved equally conclusively (or with equal ignorance) that this is not so. The truth is that neither knows.

Table 7, as given in the paper, might easily be misinterpreted either because of lack of thought or wilfully. There, it is shown that about 90% of the total investment in intercity transportation facilities in the United States is for railways and highways, being divided about equally between the two. The point to be noted, however, is that the railway plant is purely a transportation machine whereas the highways are in a fairly large degree ways of access to lands and, in not a few instances, adjuncts of recreation.

It should be emphasized also that railways perform a service in moving bulk freight and large numbers of passengers that cannot be moved by any other known means of transportation, and one without which the United States could not exist as a nation even in times of peace. For war, of course, their value is inestimable.

The author points out one step that may be taken—namely, the determination, with some approach to accuracy, of the amount of government aid affecting the provision of transport facilities and costs of transportation. That very likely may be of value.

He suggests (see heading "Transportation Costs") that "If all forms of government aid were withdrawn from commercial transportation agencies, economic laws would soon point out which form of transportation was the most economical and satisfactory to the users." In theory this is right. In practice it is not possible.

Water transportation is mixed up with flood control. Air transport has been a development of incalculable benefit to the United States and to the world, if it is conceded that the world, and transportation in the world, must advance. The construction of highways is strictly a governmental function—national, state, county, and municipal. Highways must be built with public funds.

There remain the railways, ground between the millstones of demands of organized labor for higher wages, regulation of rates by the government and, at least to some extent, competition of subsidized highway and water transportation.

The President's Board may be able to arrive at a fair determination of costs by the various different forms of transportation but this is doubtful.

The war has shown, and will show more conclusively, the absolute need of both railways and highways in the United States. The highways will be taken care of. The users, and that means the public generally, will see to that. The railways are a different problem.

It is the suggestion of the writer that the President's Board concentrate on a problem which can be solved—the maintenance in the United States, not of a governmentally owned railway system but, of a railway system that will provide efficient service at the lowest cost consistent with financial integrity.

WILLIAM J. WILGUS,[7] HON. M. AM. SOC. C. E.[7a]—This timely and informative paper on the Nation's transportation problem comes to fill a need that has never before been so pressing. The array of pertinent facts and data is impressive. To them, however, it would seem well to add others essential to the making of a sound diagnosis, as does the physician in searching for all the symptoms of a bodily ailment before announcing its causes and the measures, though painful, then to be taken for its cure. The age-old Delphic admonition, know thyself, is to be heeded in this if genuine success is to attend the effort.

The physical and financial aspects of the problem are not alone to be considered. There is also a psychological aspect that plays an enormous part in the interplay of forces bearing on the movement of persons and property from place to place. Man, by his very nature, prefers to do things for himself when thereby he may gain convenience, time, pleasure, and independence, as in the use of the motorcar. Then, too, there is his inborn urge to escape from what he deems to be the exactions of corporations dominated by unconscionable "high finance," so recently revealed at Washington.[8] Again there is his feeling that his products often may be moved to markets by highway and waterway at rates he can afford to pay, when they would remain immobile were there no such escape from shipment by rail at higher rates. In this attitude he is ignoring the countervailing fact that he must look to the railroad for never-failing service when its competitors are locked in ice and snow. Then he shrinks both from the subsidizing of tax-burdened railroads, controlled as they are by the "high finance" he fears, and the withdrawal of aids to their rivals with resulting higher rates to be borne by him, even though he recognizes the unfairness of favoring one and not the other. Finally, there is his conviction that, were it not for competition offered him by highway, waterway, airway, and pipe line, he would be powerless to secure by rail the rates and service he now enjoys, or is striving to obtain. The argument that through taxation he is paying, indirectly, additional sums for these advantages is to him an abstraction that "leaves him cold," as does the other argument, well founded though it is, that the increasing diversion of his traffic from the railroads to their subsidized competitors will in the end land the railroads, exhausted, in his lap. These are human attitudes that cannot be "poohpoohed." They have had a tremendous effect on the development of transportation and will so continue. They are among the causes that must be given weight.

[7] Weathersfield, Ascutney, Vt.

[7a] Received by the Secretary January 16, 1942.

[8] Senate Rept. No. 25, 76th Cong., 1st and 3d Sessions, Pts. 1 to 27, inclusive, and Senate Rept. No. 26, 77th Cong., 1st Session Pts. 1 and 2.

From the physical and financial angle the situation is no less difficult. The railroads—backbone of the Nation's transportation—are looked upon statistically as a national system, as they are in a physical sense. In this view the public is prone to be led astray. Though interconnected, with traffic flowing over them uninterruptedly to and from all parts of the United States, they are in truth separately owned and controlled by a multitude of corporations having diverse interests and varying in their financial status from the wofully weak to the excessively strong. Collective statistics for all of them are not applicable individually to their widely differing members. Their lack of managerial unity may be compared with the original thirteen states before they were federalized in 1789. They are without a compelling head to direct their efforts impartially for the common good—all for one and one for all—the strong to assist the weak, each being equally necessary for the public good. In the absence of unity their development as a whole has been checked, while their rivals are advancing. Highways are being extended and improved on a vast scale; waterways, often unworthy products of the malodorous "pork barrel," are "on the march"; and airways are in the picture with promise of great things to come. As to pipe lines, sight should not be lost of those devoted to the movement of natural gas and powdered substances in fluid suspension, as well as oil and gasoline mentioned by the author; nor should the effect of electric transmission lines on the movement of rail-borne coal be ignored.

In considering the development of transportation affecting the redistribution and relocation of industry, internal and foreign trade, and the Nation's safety in time of war, all of these interrelated means of movement—railroads, highways, waterways, airways, pipe lines, and transmission lines—should, of course, be given heed. The railroads, in particular, should be given consideration in their straightened circumstances.

A glance at the development of Class I railroads will disclose that their property investment from 1916 to 1940 (each of these years preceding the entry of the United States into war) increased from 17.6 billion dollars to 26.3 billion dollars, or nearly 50%, and the stockholders' equity in the hands of the public, in excess of the funded debt, from, say, 8 billion dollars to 16 billion dollars, or 100%. Corresponding to this, net income available for dividends and surplus decreased from 647 million dollars to 189 million dollars, or 71%, while cash actually disbursed in the form of dividends to stockholders with which to buy their bread and butter decreased from 306 million dollars, or 4.4%, to 159 million dollars, or 2%. On their equity, including the additional 8 billion dollars of "venture capital" which they had "plowed back" and otherwise put into their properties, their dividends dropped from something like 4% to a little more than 1%. Even these meager dividends were not earned in the 9 years from 1931 to 1939, inclusive, during which the deficits thus incurred aggregated something like a billion dollars.

The results of this unhappy picture are reflected in the money market where the railroads' aggregate property investment in excess of 26 billion dollars, represented by a net capitalization outstanding in the hands of the public totaling 17.6 billion dollars, has a quoted value of the order of, say, 10 billion dollars. In the face of this decay in values it is easy to understand

why pre-depression gross capital expenditures on Class I railroads, averaging upwards of 800 million dollars per annum, dropped to an annual average of 276 million dollars during the 10-yr period from 1931 to 1940, inclusive. The plant as a whole, therefore, could not hold its own in a sharply competitive field in which a billion dollars of new railroad investment annually, according to competent authority, was deemed necessary for that purpose. Between 1916 and 1940 total railway line mileage has retreated from 254,037 to 233,670. In the service of Class I railways, locomotives have decreased in number from 61,332 to 41,721, although their tractive effort has remained the same; freight cars from 2,253,233 to 1,653,663, and in tons capacity from 92,280,335 to 82,722,361; and passenger cars from 52,179 to 37,817. From 1930 to 1940 maintenance was neglected, perforce, and modernizing was put aside—other than on what may be termed the luxury routes. The much more extensive mileage of lesser main lines and branches patronized by an inarticulate public has borne the brunt of deprivation.

Despite competent management as manifested by a remarkable growth in the efficiency of operation, the development of railroad transportation therefore has not been upward; nor are the results of the present war-time boom to be taken as the mark of a permanent change for the better. In the words of Ernest E. Norris, president of the Southern Railway Company:[9]

> "We can foresee the thin trickle of traffic that will flow from post-war readjustment. We can divine the savage competition that the railroads will face for every ton of that traffic. And, without stretching our imagination very far, we can visualize the devastating effects of these two certainties. We can see revenues declining; expenses frozen; profit a dimming memory; capital stoutly—and rightly—refusing to invest in an industry faced with government-nursed competition."

In this it should not be forgotten that, although the owner has received a constantly lessening return on his vastly enlarged investment, the other interests involved (managements, including the interests that control them; labor; shippers; and the general public) have had increasing rewards. As has been said by an authority in the railroad field:[10]

> "Railway management cannot truly boast it is really doing a good job as long as only about one-sixth as much net income is being earned for railway stockholders as was earned for them in the decade before the present depression."

As the evidence clearly points to the impossibility of railway management doing the "good job" with no "prince" in sight to rescue its "Cinderella," the conclusion is inescapable that the railroads in their entirety cannot finance the necessities essential to their continuance as a quasi-public enterprise in private hands. In the words of the foregoing authority:[11]

> "If the United States should begin to climb out of the slough of depression and unemployment in which it has so long reposed, large additions

[9] "Railroad Data" from the Eastern Railroad Presidents Conference, November 21, 1941.
[10] Editorial in the *Railway Age*, July 20, 1940.
[11] "The Week at a Glance," *ibid.*

> to railroad plant and equipment would be needed, and 2 per cent isn't enough earnings to attract adequate capital. The railroads need a new program to supplement their 'ready' theme; and one which will hold out some hope to railway owners. Why should these latter wish to avoid government ownership, if private ownership continues indefinitely to give them nothing."

Now war is here, with the Nation's transportation situation in what may prove to be "a tight corner." The failure of the railroads to obtain materials constitutes "the greatest and most imminent danger that threatens the provision of adequate and efficient railroad service during the remainder of the present emergency." The President's Board of Investigation and Research, referred to by the author, has been injected into an interterritorial rate controversy quite foreign to its purpose.[12] An unhealthy increase in passenger and freight rates is imminent. Demands for trained personnel, materials, and equipment on an enormous scale are in sight for service overseas, in respect of which the United States is totally unprepared.

The writer, under the circumstances, sees no reason to change the views he expressed in 1939.[13] They point to a way in which, as he believes, the development of railroad transportation may be checked in its dangerous downward course and moved upward as befits the times. The day has gone by for more investigations, further reports, and lengthy studies when Fate is knocking at the door. Action is what is needed, and that quickly. Much responsibility for this action lies in the hands of the newly appointed Director of Defense Transportation.

Correction for *Transactions:* In December, 1941, *Proceedings*, Table 3(*c*), page 1836, delete the $ sign on Item No. 10.

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[12] "The Week at a Glance," *Railway Age*, January 10, 1942.

[13] "Nationalization of the Railroads?" by William J. Wilgus, *Civil Engineering*, October, 1939, p. 593.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

DESIGN AND CONSTRUCTION OF SAN GABRIEL DAM NO. 1

Discussion

BY JACOB FELD, M. AM. SOC. C. E.

JACOB FELD,[6] M. AM. SOC. C. E.[6a]—A wealth of data on granular materials and rock fills is contained in this paper. Results of field experience and laboratory tests are correlated with each other and also with actual results in the construction work. The paper merits intensive study not only by the practical engineer who directs the performance of work but even more by the laboratory technicians, because the author clearly describes the type of information needed in the design and construction of rock-fill dams.

In works of such magnitude, the honest and commendable example set by this project, of revising the design after the field organization has had sufficient experience with the materials involved to permit true evaluation of the assumptions upon which the design was based, should be seriously followed. The mathematical development of the design can be beyond any criticism, and the design can be approved "by consulting engineers and geologists of national repute"; yet all of it is based on assumptions of what the materials will be and how they will act. It has been the writer's feeling that no amount of laboratory tests on soils, whether natural or (as in this project) artificial, can give a complete quantitative answer. The best that can be expected is a determination of the limits within which the properties of the soil may range. Such limits may be far apart, and designs of the structure for each limit may be even further apart. Averaging the laboratory results and using such data for an "average" design is not honest, economical, or even safe. When the possible range is large enough, why cannot alternate designs be prepared, or, for a project of this magnitude, several designs, to be adjusted as the true character and action of materials encountered are learned?

Economical moving of rock by blasting as in excavation work is not always economical rock production in quarry work. As a matter of fact, most economical excavation work is often dependent upon the size of the rock produced,

NOTE.—This paper by Paul Baumann, M. Am. Soc. C. E., was published in September, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1941, by E. Soucek, Jun. Am. Soc. C. E.; December, 1941, by William P. Creager, M. Am. Soc. C. E.; and January, 1942, by Joseph Jacobs, M. Am. Soc. C. E.

[6] Cons. Engr., New York, N. Y.

[6a] Received by the Secretary January 14, 1942.

where mucking equipment is limited by space requirements. In the excavation of a rock tunnel portion of the Sixth Avenue Subway in New York, N. Y.,[7] a slower acting dynamite was used to prevent a seamy schist rock from breaking out in slabs too large for the mucking machine. In describing the quarry operation, Mr. Baumann states that only one fourth of the rock was of proper size. There is a description of the type of quarrying and the kind of explosive used, and the writer would like to ask whether tests were made with other explosives and different drilling methods. Certainly, the quarrying of 10,000,000 cu yd of rock deserved a little study.

The tests of consolidation and percolation for different mixtures of fine and coarse materials are of great value. From the data as summarized in Fig. 4 and Table 2, two conclusions can be reached:

(1) The mixture of maximum density does not provide maximum resistance to seepage, because for similar materials the shape of the channels formed by the voids, and not the total percentage of voids, is the chief factor of percolation; and

(2) The mixture of maximum initial density shows the minimum change in density under load, chiefly because the more intimate contact of grains in the mixture of maximum density reduces local overstresses and thereby reduces edge and point crushing within the mass.

The author's attempt to evaluate the shear coefficient by measuring the total earth pressure within a small cylinder is subject to several objections. The validity of the formula for enclosed shallow bins is one objection; another is the variations in total pressure with volume expansion of the retaining cylinder. The direct shear test seems a preferable method for evaluating the shearing coefficient.

The complete coverage of the subject of compaction under various moisture contents, as well as with different equipment, is a valuable contribution. It is noteworthy that saturation practically eliminates breakdown of the grains, whereas the maximum effect of compaction equipment can be obtained when the moisture is between 10% and 15% of the dry weight. The explanation is probably in the tensile strength provided by the moisture films in the granular mass.

That even field tests will not provide accurate estimates of consolidation settlement is shown by Fig. 30, which reports an observation of the actual consolidation of about half the value expected from cylinder tests. One explanation of the discrepancy is in the difference of time during which the same unit loads were applied, as well as the lateral restraint found in the actual dam fill as contrasted with the artificial cylinder restraint with less frictional resistance along the boundary of the test sample inside the cylinder. The action of the completed dam in deflecting under full water load with perfect rebound shows that the structure is substantially elastic in nature; no better test of the excellence of design and performance could be expected.

Corrections for *Transactions:* In September, 1941, *Proceedings*, page 1219, last line, insert after "horizontal pressure" the parenthetical explanation "(in thousands of pounds per square inch)." See also corrections cited in *Proceedings*, January, 1942, p. 186.

[7] "Tunnel Construction, Sixth Avenue Subway, New York, N. Y.," by Jacob Feld, *Proceedings*, Am. Soc. C. E., April, 1941, p. 617.

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DISCUSSIONS

TIMBER FRICTION PILE FOUNDATIONS

Discussion

BY E. H. CONNOR, M. AM. SOC. C. E.

E. H. CONNOR,[4] M. AM. SOC. C. E.[4a]—The tests of the supporting value of piles in various groups, as reported by Mr. Masters, are worth many times their cost, and the formulas deduced and results of calculations therefrom reveal many interesting facts. Many more tests should be made and could be made with a saving of millions of dollars from the information gained: Tests on the spacing of piles in groups, on the result of leaving out center piles of a group, on the supporting values in sand in the river bed, on the value of foundation of piles driven on a batter, etc.

Some engineers specify that a jet cannot be used in driving. Why? It would save money in many cases. Do they know that in a week's time one pile driven in sand or clay would support a lesser load than the other.

It is evident that many pile-foundation designs are made on the assumption that 60 piles will carry 60 times as much as one pile, and that it is economical to crowd these piles into as small an area as possible. The result is a spacing so close that the piles are very difficult and expensive to drive.

Of course, the volume and the cost of the footing course of concrete are reduced. The strength of the foundation may be reduced still more. A proper spacing of piles to secure the same strength would increase the cost of concrete but reduce the cost of piles. The economical design then becomes a question of unit costs of concrete and piles, length of piles, thickness of concrete base or sealing course. The cost of cofferdam and excavation also must be considered; but the proper design now becomes a scientific problem. It is still not an exact science.

The writer often has thought that the perimeter of the entire group in a certain location determined the power of the group rather than the number of piles, if the piles were not spaced farther apart than, say, 4 ft. For instance (see Table 1(c)), a group of nine piles T11–19 failed under a load of $(56.3 \times 9 =)$ 507 tons. Assuming that the perimeter of group length is $(2 \times 4 + 1) \times 4$

NOTE.—This paper by Frank M. Masters, M. Am. Soc. C. E., was published in November, 1941, *Proceedings*.

[4] Vice-Pres. and Cons. Engr., Mo. Val. Bridge & Iron Co., Leavenworth, Kans.

[4a] Received by the Secretary December 15, 1941.

$\times$ 59.7 = 2,149 sq ft, the shear is $\frac{507}{2,149}$ = 0.236 tons per sq ft. The 16-pile group T37 to 52 failed under load of (42.9 $\times$ 16 =) 686 tons. The perimeter of the group multiplied by the penetration is (3 $\times$ 4 + 1) $\times$ 4 $\times$ 60 = 3,120 sq ft, and the shear is $\frac{686}{3,120}$ = 0.220 tons per sq ft—a difference of only 7%, which amount could easily exist in two similar groups of the same number.

How much would the group of sixteen piles have carried if spaced on 3-ft centers? How much would this group have carried if the four interior piles had been omitted?

Another feature has come to the attention of many. By standing unloaded a few days after driving a pile will carry an increased load. Four piles were driven in San Francisco Bay to support a derrick. They were 90 ft long and penetrated about 25 ft of water and 55 ft of clay. They drove easily and did not show greatly increased resistance to driving as the penetration increased. It was suggested that the length of the piles be increased to 120 ft. After waiting two days, a single-acting steam hammer (5,000-lb ram, 36-in. stroke) could not move the 90-ft piles. These lengths were used for a year without settlement.

In 1900, the writer's firm built a creosoted pile trestle across Biloxi Bay. The piles drove easily in the mud—too easily. They would not support much more than a heavy hammer. The contractor was alarmed. The specifications required that the causeway carry a load of 100 lb per sq ft. It was completed and several panels loaded to 100 lb without any settlement and the causeway served the public nearly thirty years.

In 1912 the Kansas City Terminal Railway Company made tests of concrete piles driven in dry loam, testing them immediately after driving. Some were tapered; some had parallel sides. The tapered piles carry considerably more than those with parallel sides but they are difficult to keep plumb and they yield more readily to side movement from obstructions struck while driving.

From the contractor's point of view, one other point in spacing piles should be given more consideration; that is, the distance from the outside edge of the concrete base to the center of the outside row of piles. This is usually shown on the plans as 18 in. In a deep excavation or cofferdam, wales at least 12 in. wide are needed. A 5,000-lb, single-acting steam hammer (36-in. stroke) has an over-all clearance of 40 in. requiring 36 in., with 4-in. clearance, from the center of the outside pile to the inside of sheet piles unless a follower is used.

Correction for *Transactions:* In November, 1941, *Proceedings*, on page 1667, denominator of Eq. 2*c*, change "$(r^2 - Z^2)^{1.5}$" to "$(r^2 + Z^2)^{1.5}$"; and, on page 1668, Eq. 4*b* (last quantity in the first line), change "$+ R_c - 1$" to "$+ (R_c - 1)$."

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DISCUSSIONS

LATERAL STABILITY OF UNSYMMETRICAL I-BEAMS AND TRUSSES IN BENDING

Discussion

BY ROBERT K. SCHRADER, JUN. AM. SOC. C. E.

ROBERT K. SCHRADER,[9] JUN. AM. SOC. C. E.[9a]—Two types of loading commonly encountered in practice—(*a*) uniformly distributed load, and (*b*) two equal loads symmetrically located on the beam—lend themselves readily to analysis by the method presented in this paper. In his treatment of this problem, herein, the writer leaves the directions of the coordinate axes unchanged, but the origin is transferred to the center of the undistorted beam.

(*a*) *Uniformly Distributed Load.*—With the coordinate system thus transferred, the author's approximate expression for the angular deformation, Eq. 1, now becomes:

$$\phi = a \cos \frac{\pi x}{L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

Eq. 5, for the strain energy W_e, remains unchanged except that the limits of integration now extend from $-\frac{L}{2}$ to $+\frac{L}{2}$. For uniformly distributed load the vertical bending moment M to be substituted in Eq. 8 is

$$M = \frac{q}{2}\left(\frac{L^2}{4} - x^2\right) \ldots\ldots\ldots\ldots\ldots\ldots (49a)$$

Consequently,

$$\left(\frac{d^2y}{dx^2}\right)^2 = \frac{q^2}{4\,E^2\,I^2}\left(\frac{L^4}{16} - \frac{L^2\,x^2}{2} + x^4\right)\phi^2 \ldots\ldots\ldots\ldots (49b)$$

Substitution of Eqs. 1 and 49*b* in the equation for the strain energy yields, for this case,

$$W_e = \frac{a^2}{4\,I}\left[\frac{q^2\,L^5\,(\pi^4 + 45)}{120\,\pi^4\,E} + \frac{\pi^4\,h^2\,I_1\,I_2\,E}{L^3} + \frac{\pi^2\,C\,E\,I}{L}\right] \ldots\ldots (50)$$

NOTE.—This paper by George Winter, Esq., was published in December, 1941, *Proceedings*.

[9] 2d Lt., 57th Coast Artillery, U. S. Army, Camp Pendleton, Va.

[9a] Received by the Secretary January 5, 1942.

The loss in potential energy, or the work done by the load, is equal to the magnitude of the load multiplied by the distance the load falls as the beam deflects laterally. This distance is again divided into two components, Δ_1 and Δ_2 (see Fig. 2). It is seen that for any section

$$\Delta_1 = (f - c_2)(1 - \cos \phi) \ldots\ldots\ldots\ldots\ldots\ldots (51a)$$

Substituting c_2 from Eq. 3*b*, expanding $\cos \phi$ in a power series in ϕ, and neglecting all terms beyond the second in this expansion, one obtains

$$\Delta_1 = \frac{(f I - h I_1)}{2 I} \phi^2 \ldots\ldots\ldots\ldots\ldots\ldots (51b)$$

From Fig. 7 it is seen that

$$\Delta_2 = \Delta'_2 - \Delta''_2 \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

in which Δ'_2 is the vertical displacement of the flexural center at midspan due to lateral displacement, and Δ''_2 is the difference between this displacement

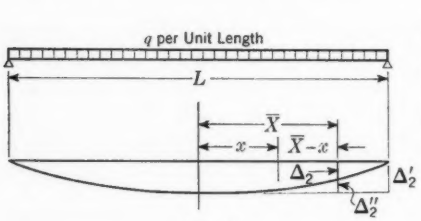

FIG. 7

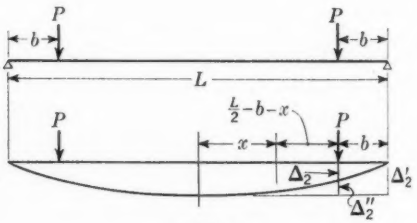

FIG. 8

and that of a section at a distance $\bar{x}$ from the origin. From the geometry of the problem the magnitudes of these two components are given by the following expressions:

$$\Delta'_2 = \int_0^{L/2} \phi \frac{d^2y}{dx^2} \left(\frac{L}{2} - x \right) dx \ldots\ldots\ldots\ldots\ldots\ldots (53a)$$

and

$$\Delta''_2 = \int_0^{\bar{x}} \phi \frac{d^2y}{dx^2} (\bar{x} - x)\, dx \ldots\ldots\ldots\ldots\ldots\ldots (53b)$$

The total work done by the uniformly distributed load, q, is

$$W = \int_{-L/2}^{L/2} q\, (\Delta_1 + \Delta_2)\, dx \ldots\ldots\ldots\ldots\ldots\ldots (54a)$$

By substitution of the expressions for Δ_1 and Δ_2 given in Eqs. 51 and 52 and by evaluation of the integrals, the total work becomes

$$W = \frac{a^2}{4 I} \left[q\, L\, (f I - h I_1) + \frac{q^2 L^5 (\pi^4 + 45)}{60\, \pi^4 E} \right] \ldots\ldots\ldots (54b)$$

The critical load is obtained by equating W_ϵ from Eq. 50 to W from Eq. 54*b*,

which results in

$$q_{cr} = \frac{60\,\pi^4\,E}{L^4\,(\pi^4 + 45)}\left[\,(h\,I_1 - f\,I) + \sqrt{(h\,I_1 - f\,I)^2 + \frac{\pi^4 + 45}{30}\left(h^2\,I_1\,I_2 + \frac{C\,I\,L^2}{\pi^2}\right)}\,\right] \quad \ldots\ldots (55)$$

For the special case of a symmetrical I-beam loaded along its gravity axis, the results from Eq. 55 coincide with those obtained by Professor Timoshenko for this case.[10]

(*b*) *Two Equal Loads, Symmetrically Placed.*—Considering the right half of the beam shown in Fig. 8, it is seen that, from $x = 0$ to $x = (L/2 - b)$, the bending moment is

$$M = P\,b \quad \ldots\ldots (56a)$$

and therefore, for this part of the beam,

$$\left(\frac{d^2y_1}{dx^2}\right)^2 = \frac{P^2\,b^2}{E^2\,I^2}\,\phi^2 \quad \ldots\ldots (56b)$$

From $x = (L/2 - b)$ to $x = L/2$ the moment is

$$M = P\left(\frac{L}{2} - x\right) \quad \ldots\ldots (57a)$$

and therefore,

$$\left(\frac{d^2y_2}{dx^2}\right)^2 = \frac{P^2}{E^2\,I^2}\left(\frac{L^2}{4} - L\,x + x^2\right)\phi^2 \quad \ldots\ldots (57b)$$

The strain energy of one half of the beam is then obtained from

$$W_e = \frac{E}{2}\left[\,I\int_0^{(L/2-b)}\left(\frac{d^2y_1}{dx^2}\right)^2 dx + I\int_{(L/2-b)}^{L/2}\left(\frac{d^2y_2}{dx^2}\right)^2 dx + h^2\,\frac{I_1\,I_2}{I}\int_0^{L/2}\left(\frac{d^2\phi}{dx^2}\right)^2 dx + C\int_0^{L/2}\left(\frac{d\phi}{dx}\right)^2 dx\,\right] \quad \ldots\ldots (58a)$$

Substitution of Eqs. 56*b* and 57*b* and evaluation of the integrals yield

$$W_e = \frac{a^2}{8\,I}\left[\,\frac{P^2}{E}\left(b^2\,L - \frac{4\,b^3}{3} - \frac{b\,L^2}{\pi^2}\cos\frac{2\,\pi\,b}{L} + \frac{L^3}{2\,\pi^3}\sin\frac{2\,\pi\,b}{L}\right) + \frac{\pi^4\,h^2\,I_1\,I_2}{L^3} + \frac{\pi^2\,C\,E\,I}{L}\,\right] \quad \ldots\ldots (58b)$$

To find the work done by each of the two forces P, the distance the load falls will be divided into two components Δ_1 and Δ_2 as before, and again

$$\Delta_1 = \frac{(f\,I - h\,I_1)}{2\,I}\,\phi^2{}_b = \frac{(f\,I - h\,I_1)}{2\,I}\,a^2\sin^2\frac{\pi\,b}{L} \quad \ldots\ldots (59)$$

[10] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., Table 23, p. 268.

in which ϕ_b is the angular displacement of the load. From Fig. 8 it is seen that, in analogy with Eq. 52, $\Delta_2 = \Delta'_2 - \Delta''_2$, as before, in which Δ'_2 is again the vertical displacement of the flexural center at midspan, and Δ''_2 is now the difference between this displacement and that at the loaded section. Consequently,

$$\Delta'_2 = \int_0^{(L/2-b)} \phi \frac{d^2y_1}{dx^2}\left(\frac{L}{2} - x\right) dx + \int_{(L/2-b)}^{L/2} \phi \frac{d^2y_2}{dx^2}\left(\frac{L}{2} - x\right) dx \ldots (60a)$$

and

$$\Delta''_2 = \int_0^{(L/2-b)} \phi \frac{d^2y_1}{dx^2}\left(\frac{L}{2} - b - x\right) dx \ldots\ldots\ldots\ldots (60b)$$

The work done by P is then

$$W = P\,(\Delta_1 + \Delta_2) \ldots\ldots\ldots\ldots\ldots\ldots (61a)$$

By substitution of Eqs. 59 and 60, and by evaluation of the integrals, the work becomes

$$W = \frac{a^4}{8\,I}\left[4\,P\,(f\,I - h\,I_1)\sin^2\frac{\pi\,b}{L} + \frac{P^2}{E}\left(2\,b^2\,L - \frac{8\,b^3}{3} - \frac{2\,b\,L^2}{\pi^2}\cos\frac{2\,\pi\,b}{L} + \frac{L^3}{\pi^3}\sin\frac{2\,\pi\,b}{L}\right)\right] \ldots . (61b)$$

The critical load is obtained by equating W_e from Eq. 58*b* to W from Eq. 61*b*, which results in

$$P_{cr} = A\,\frac{E}{L^3}\left[(h\,I_1 - f\,I) + \sqrt{(h\,I_1 - f\,I)^2 + B\,h^2\,I_1\,I_2 + D\,C\,I\,L^2}\right] .. (62)$$

In Eq. 62, A, B, and D are numerical coefficients the values of which, for different ratios of b/L, are given in Table 1.

TABLE 1.—NUMERICAL COEFFICIENTS FOR THE SOLUTION OF EQ. 62

Coefficients	Ratios b/L									
	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
A	19.59	19.20	18.64	18.00	17.30	16.62	16.00	15.51	15.09	14.93
B	101.6	26.57	12.68	7.84	5.63	4.53	3.83	3.48	3.31	3.27
D	10.3	2.69	1.28	0.794	0.570	0.448	0.369	0.353	0.335	0.331

It may be ascertained easily that, for $b/L = 0.5$, the value $2\,P_{cr}$ of this analysis is identical with that for a concentrated center load obtained by the author. Also, for $b/L = 0.05$, almost the entire length of the beam is subject to pure bending. The critical moment so computed differs from that obtained by the author for pure bending of the entire beam by only 0.1%.

Acknowledgment.—This discussion is based on a thesis prepared under the supervision of the author and submitted to Cornell University, at Ithaca, N. Y., in 1941 in partial fulfilment of the requirements for the degree of Master of Civil Engineering.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

BENDING MOMENTS IN THE WALLS OF RECTANGULAR TANKS

Discussion

BY I. K. SILVERMAN, ASSOC. M. AM. SOC. C. E.

I. K. SILVERMAN,[13] ASSOC. M. AM. SOC. C. E.[13a]—Even with such a tractable ratio of $\frac{b}{a}$ equal to unity, the amount of calculation involved in the solution offered by the author is very great; and it is probably for this reason that Professor Young resorted to the "roughing in" of the curves denoted by 5(*a*) in Figs. 6 and 7. For determining the character of these curves, a single control point and two limiting curves were available.

Because of the amount of computation involved, any approximate solution that will give sufficiently accurate values with less computation is to be welcomed. Such an approximate solution is available[14] and the relative accuracy of the two can be compared. Unfortunately the Marcus approximate method was applied to the ratio $\frac{b}{a} = 0.5$ for case (*a*), Fig. 5, and the results for this case must be compared with values from Curves 5(*a*) of Figs. 6 and 7 which are approximate. Moreover, the Marcus method was applied to case (*b*), Fig. 5, and Table 2 shows the values obtained by Marcus for cases (*a*) and (*b*) compared with those taken from the curves of Figs. 6 and 7.

TABLE 2.—COMPARISON OF C FOR $b/a = 0.5$

Method	C_1		C_2	
	Fig. 5(*a*)	Fig. 5(*b*)	Fig. 5(*a*)	Fig. 5(*b*)
Young....	0.102	0.064	0.049	0.036
Marcus...	0.082	0.063	0.042	0.039

It is seen that the results of the author's method and the Marcus approximate solution agree fairly well for case (*b*), but there is quite a discrepancy in the values given for case (*a*), Fig. 5. Since the Young values are taken from a curve that is "roughed in," no definite statement can be made concerning the accuracy of the Marcus method until a mathematically exact solution is made

NOTE.—This paper by Dana Young, Assoc. M. Am. Soc. C. E., was published in November, 1941, *Proceedings*.

[13] Denver, Colo.

[13a] Received by the Secretary December 18, 1941.

[14] "Beitrag zur Untersuching von Behältern mit ebeben Wandungen," by H. Marcus, *Der Bauingenieur*, February 7, 1936, p. 40; also "La Résistance des Réservoirs à Parois Planes," by the same author, *Annales des Ponts et Chaussées*, April, 1937, p. 540.

of curves 5(*a*), Figs. 6 and 7, using the Young method. However, the comparison of the values for case (*b*) would seem to indicate that the Marcus method can be relied upon. Another distinguishing feature is that the Marcus solution for $b/a = 0.5$ shows that the maximum value of M_x occurs at $x = a/2, y = b/4$, and its value is $0.050\,\gamma\,b^3$

As the author states, the solution given in the paper can be applied to counterforted walls subjected to hydrostatic pressure by assuming constant thickness and fixity at the base and counterfort. An interesting problem is the analysis of the part of the counterforted wall, ABCD (Fig. 8(*a*)). Not

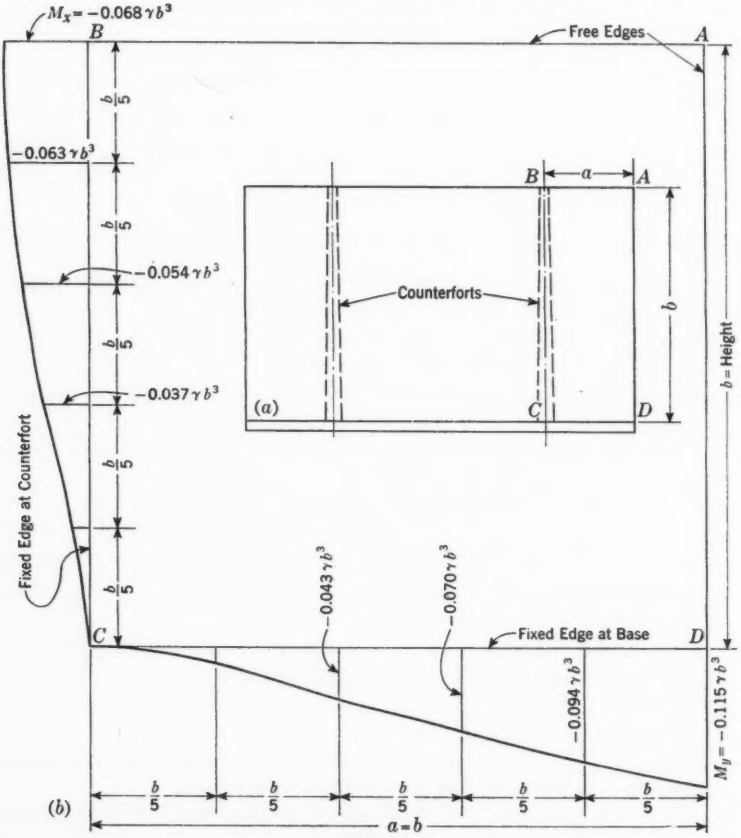

FIG. 8

only are the questions of moments and shears of interest, but the following question may be posed: What must be the ratio of b/a so that the deflection of line AD shall be sensibly the same as that of a cantilever of the same height?

The writer has applied the Marcus method to such a plate ABCD of Fig. 8(*a*) with b/a = unity and obtained results as shown in Fig. 8(*b*). For ratios $b/a < 1$ the Marcus method is not satisfactory when applied to plates ABCD of Fig. 8(*a*), and Marcus states that the computations become tedious for $b/a < 0.5$ when applied to walls of cases (*a*) and (*b*), Fig. 5.

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DISCUSSIONS

THE SUSPENSION BRIDGE TOWER CANTILEVER PROBLEM

Discussion

BY JOHN I. COPP, ASSOC. M. AM. SOC. C. E.

JOHN I. COPP,[8] ASSOC. M. AM. SOC. C. E.[8a]—The procedure for the exact solution of tower stresses for all loading conditions that ordinarily occur in a plane parallel to the bridge axis is described in this paper. The author has also indicated some approximations that may be used with accuracy for certain loading conditions in evaluating the tower stresses and deflections. His proposed method of finding the effect of the tower dead load on the horizontal force F at the tower top by "* * * assuming the tower to take the shape of some well-known curve such as the parabola" (see heading "Discussion of Numerical Applications") has been used by the writer and found to give a very close approximation to the true value.

When preliminary investigations are being made for planning cable-spinning operations or tower erection, it is of value to have some rapid means of approximating the tower deflections and stresses prior to the final analysis by the exact method. By applying the foregoing parabolic assumption, it is comparatively simple to derive the approximate equations for any and all loading conditions that are indicated by the author in his various cases. The assumption of a parabolic curve for the tower eliminates the solution of all differential equations and the determination of the constants of integration when deriving the equations.

The writer believes that as a supplement to the author's formal solution of the various cases the method of deriving the approximate equations would be of value. The derivation of the equations for a few typical loading conditions is given herein with some numerical applications by the exact and approximate methods for comparison.

In the derivation of the approximate equations the author's notation will be used in so far as it is applicable with additions as required:

NOTE.—This paper by Blair Birdsall, Assoc. M. Am. Soc. C. E., was published in April, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Francis P. Witmer, M. Am. Soc. C. E.

[8] Senior Designer, Bethlehem Steel Co., Fabricated Steel Constr., Bethlehem, Pa.

[8a] Received by the Secretary January 17, 1942.

h = tower height, in feet;
Δ' = initial tower-top deflection for a free standing tower, in feet;
Δ = final tower-top deflection, in feet;
δ_n = deflection increment of tower top (caused by tower moments that are functions of the tower curve), in feet;
$\Sigma\delta$ = total tower-top deflection (caused by tower moments that are functions of the tower curve), in feet;
R = known vertical load at tower top, in pounds;
F_Δ = horizontal force at tower top to maintain a known horizontal deflection Δ, in pounds;
F = a horizontal force at tower top, in pounds;
E = modulus of elasticity for the tower, in pounds per square inch;
e = eccentricity of the vertical load R with respect to the center line at tower top, in feet;
M_x = bending moment in tower at a distance x from the tower top, in foot-pounds; and
w = dead load of tower per linear foot of height (assumed uniform).

Free Standing Tower (*Total Deflection of Tower Top Due to the Dead Load of Tower*).—Assume the tower curve $T\ A\ B$ (Fig. 8(*a*)) to be a parabola with its vertex at point B and with an initial deflection Δ' (Fig. 8(*b*)) resulting from any tower moments that are not functions of the tower curve.

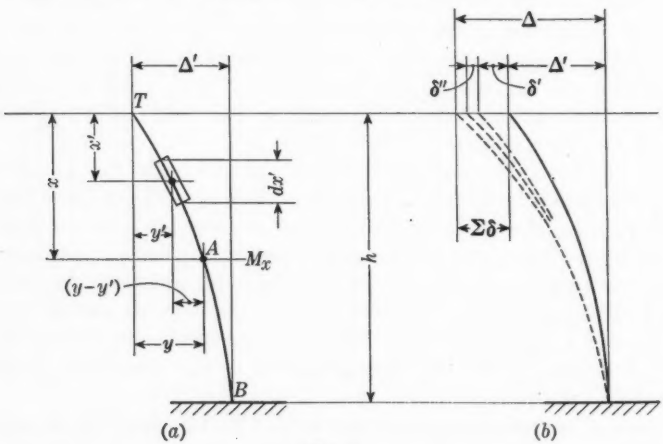

FIG. 8

The total deflection $\Sigma\delta$ (Fig. 8(*b*)) due to the dead load of the tower is determined by taking the sum of a geometrical series, since the deflection δ' will induce an additional deflection δ'' and an infinite number of deflection increments will result with a constant ratio between successive increments—that is, $\frac{\delta'}{\Delta'} = \frac{\delta''}{\delta'}$ = etc. The first term of this series, δ', is determined by taking moments of the $\frac{M}{E\,I}$-diagram, for the tower dead load, about the tower top.

The formula for the sum of a geometrical series is

$$s = \frac{a\,(r^n - 1)}{r - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

which, applied to the tower deflection increments, gives a = first term = δ'; r = ratio for the series = $\frac{\delta'}{\Delta'}$; n = number of terms = ∞; s = sum of the series = $\Sigma\delta$; and the sum of the series then will be

$$\Sigma\delta = \frac{\delta'\left[\left(\frac{\delta'}{\Delta'}\right)^{\infty} - 1\right]}{\frac{\delta'}{\Delta'} - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

In Eq. 31 the ratio $\frac{\delta'}{\Delta'}$ must be less than unity (if the critical loading for the tower has not been reached); hence

$$\lim_{n=\infty}\left(\frac{\delta'}{\Delta'}\right)^n = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

and the equation for deflection will be

$$\Sigma\delta = \Delta'\left(\frac{\delta'}{\Delta' - \delta'}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

In Eq. 33 the value of δ' is equal to the moment of the $\frac{M}{EI}$-diagram, about the tower top—

$$\delta' = \int_{x=0}^{x=h} \frac{M_x\,x}{E\,I}\,dx \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

in which the value of M_x, the moment at the point x, due to the dead load of the tower above point x, for a parabolic curve, can be shown to be

$$M_x = \frac{\Delta'\,w}{h}x^2 - \frac{2}{3}\frac{\Delta'\,w}{h^2}x^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Substituting this value of M_x in Eq. 34 and evaluating the integral—

$$\delta' = \frac{1}{E\,I}\int_{x=0}^{x=h}\left(\frac{\Delta'\,w}{h}x^2 - \frac{2}{3}\frac{\Delta'\,w}{h^2}x^3\right)x\,dx = \Delta'\left(\frac{7\,w\,h^3}{60\,E\,I}\right) \quad \ldots\ldots (36)$$

The deflection for the free standing tower due to its own dead load is then found by substituting the value of δ' from Eq. 36 in Eq. 33

$$\Sigma\delta = \Delta'\left(\frac{7\,w\,h^3}{60\,E\,I - 7\,w\,h^3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

The value of Δ (Fig. 8(*b*)) then will be

$$\Delta = \Delta'\left(1 + \frac{7\,w\,h^3}{60\,E\,I - 7\,w\,h^3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

Tower with Controlled Deflection.—When the tower-top deflection is controlled by a known amount Δ', as in cable-spinning operations, the reduction in the horizontal force F (say F') due to the dead load is found by equating the deflection caused by a single horizontal force F' to the deflection δ' and solving for F'; that is,

$$\frac{F' h^3}{3 E I} = \Delta' \left(\frac{7 w h^3}{60 E I} \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

and

$$F' = \Delta' \left(\frac{7}{20} w \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

Applying this reduction, F', to a tower with a controlled deflection Δ', the horizontal force F is determined by deducting, from the force required to deflect the weightless tower, an amount Δ', the force F'.

The force required to deflect the weightless tower an amount Δ' is

$$F_\Delta = \frac{\Delta' 3 E I}{h^3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

Hence, the net force will be

$$F = \frac{\Delta' 3 E I}{h^3} - \Delta' \frac{7}{20} w \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42)$$

in which, as previously stated, Δ' is an initial deflection of the tower top resulting from moments that are not functions of the tower curve.

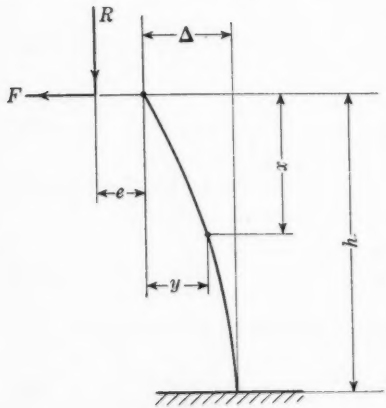

FIG. 9

Free Standing Tower with Horizontal and Eccentric Vertical Loads at Tower Top.—In Fig. 9 the moment at any point x, y will be

$$M_x = F x + R (e + y) \ldots\ldots (43)$$

which may be considered as the sum of three moments, $F x$, $R e$, and $R y$. The moments $F x$ and $R e$ are independent of the tower curve, whereas the moment $R y$ is a function of the tower curve. The deflection resulting from the moment $R y$ must be treated as the sum of a geometrical series, for a free standing tower, as was done in the case for the dead load of the tower. The deflection due to the horizontal force F will be

$$\Delta_F = \frac{F h^3}{3 E I} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44a)$$

The deflection due to the moment $R e$ will be

$$\Delta_{Re} = \int_{x=0}^{x=h} \frac{R e x}{E I} dx = \frac{R e h^2}{2 E I} \ldots\ldots\ldots\ldots\ldots\ldots (44b)$$

The tower-top deflection due to these two moments then will be

$$\Delta' = (\Delta_F + \Delta_{Re}) = \frac{F\,h^3}{3\,E\,I} + \frac{R\,e\,h^2}{2\,E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

which is the initial deflection for determining the deflection caused by the moment $R\,y$.

The deflection due to the moment $R\,y$ is determined by the moment of the $\frac{M}{E\,I}$-diagram about the tower top:

$$\delta' = \int_{x=0}^{x=h} \frac{R\,y\,x}{E\,I}\,dx \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

in which

$$y = \frac{\Delta'}{h^2}(2\,h\,x - x^2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

Then:

$$\delta' = \int_{x=0}^{x=h} \frac{R\,\Delta'}{E\,I\,h^2}(2\,h\,x - x^2)\,x\,dx = \Delta'\frac{5\,R\,h^2}{12\,E\,I} \quad \ldots\ldots\ldots (48)$$

The total deflection due to the moment $R\,y$ will be equal to the sum of a geometrical series with an infinite number of terms as indicated by Eq. 33. Substituting for Δ' and δ' their values as given by Eqs. 45 and 48, respectively, in the general Eq. 33, and adding the initial deflection Δ':

$$\Delta = \left(\frac{F\,h^3}{3\,E\,I} + \frac{R\,e\,h^2}{2\,E\,I}\right)\left(1 + \frac{5\,R\,h^2}{12\,E\,I - 5\,R\,h^2}\right) \quad \ldots\ldots\ldots (49)$$

which is the total deflection for a free standing tower with a horizontal and eccentric vertical load at the tower top.

For a free standing tower with a horizontal force F and a concentric vertical load R, the value of e in Eq. 49 becomes zero and the equation for deflection will be

$$\Delta = \frac{F\,h^3}{3\,E\,I}\left(1 + \frac{5\,R\,h^2}{12\,E\,I - 5\,R\,h^2}\right) \quad \ldots\ldots\ldots\ldots\ldots (50)$$

It is of interest to note that an approximate value of the critical concentric load R, for a free standing tower, may be evaluated from the general Eq. 31. It can be shown that Eq. 31 approaches infinity as the ratio $\frac{\delta'}{\Delta'}$ approaches unity. Equating the ratio in Eq. 48 to unity and solving for R,

$$R = \frac{12\,E\,I}{5\,h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

A comparison of this approximate critical load with Euler's theoretical load shows an error of less than 3% in the approximate value.

When the tower is loaded at the top with an eccentric vertical load and with the horizontal deflection controlled, the horizontal force for equilibrium is determined by equating (for a weightless tower) the deflection resulting

from a horizontal force F' to the sum of the first increments of deflection caused by the moments $R\,e$ and the vertical load R and solving for F':

$$\frac{F' h^3}{3 E I} = \frac{R e h^2}{2 E I} + \Delta' \frac{5 R h^2}{12 E I} \quad \ldots\ldots (52)$$

or,

$$F' = \frac{3}{2}\frac{R e}{h} + \Delta' \frac{5 R}{4 h} \quad \ldots\ldots (53)$$

which gives the reduction in the horizontal force required to deflect the tower an amount Δ' with no other loads. The reduced force F resulting from the eccentric vertical load will then be

$$F = \frac{3 \Delta' E I}{h^3} - \frac{3}{2}\frac{R e}{h} - \Delta' \frac{5 R}{4 h} \quad \ldots\ldots (54)$$

In Eq. 53 the value of e may be either negative or positive, depending on whether the moment $R\,e$ decreases or increases, respectively, the moment $F\,x$ at any point in the tower. The value of e will be zero for a concentric load R. When R is concentric, the resulting horizontal force F will be

$$F = \Delta' \frac{3 E I}{h^3} - \Delta' \frac{5 R}{4 h} \quad \ldots\ldots (55)$$

Numerical Applications of the Approximate Solutions.—Refer to curve 3, Fig. 5(*a*) of the paper, as a comparable example and apply the corresponding approximate Eq. 55 with the following values: R (assumed) = 1,200,000 lb; h = 188 ft; I = 1,061.7 in.^{2} ft^2; Δ' (assumed) = 1.0 ft; E (assumed) = 29,000,000 lb per sq in.; and F = 5,922 lb.

Entering Fig. 5(*a*) with R = 1,200,000 lb, the value of $\frac{F}{1{,}000\,\Delta} = 6$, which gives a value of F = 6,000 lb and shows an error, in the approximate solution, of about 1.5% when compared to the author's comparable case.

Using the same tower properties and assuming it as free standing, the total deflection by the approximate solution (Eq. 50) is Δ = 1.013 ft, in which R, E, I, and h are as stated previously and F (from Fig. 5(*a*)) is 6,000 lb. From curve 3, Fig. 5(*a*): $\Delta = \frac{F}{6{,}000}$ = 1.00 ft, which shows an error of about 1.5% in the approximate solution.

It appears from the foregoing derivations and numerical examples that the approximate solutions may be used to advantage as an approach to the formal solutions as outlined by the author. They also might be used for the preliminary design of towers or as the first approximation if the tower stresses were to be determined by successive approximations.

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DISCUSSIONS

PILE-DRIVING FORMULAS

PROGRESS REPORT OF THE COMMITTEE ON THE BEARING VALUE OF PILE FOUNDATIONS

Discussion

BY MESSRS. KARL TERZAGHI, RALPH B. PECK, AND ARTHUR CASAGRANDE

KARL TERZAGHI,[67] M. AM. SOC. C. E.[67a]—Paragraph *A*-4 of this Report contains a derivation of a general formula (Eq. 3), but does not mention the fact that the equation contains the same energy loss twice. This is disappointing, to say the least, and bars Report *A* from incorporation in the Manual, unless it is rewritten in the light of the penetrating and conclusive mathematical analysis of the subject by A. E. Cummings,[68] M. Am. Soc. C. E. Even after revision, the contents of Report *A* belong in an Appendix, because a manual is not a textbook.

Report *B* deals with its subject strictly in accordance with the scope and purpose of the Manual. However, in the writer's opinion, it would be advisable to make some of the statements more specific and to eliminate whatever may encourage the reader to use theories and procedures that have not yet stood adequate tests of practical application. The following paragraphs contain suggestions for a partial revision of Report *B* and a presentation of the arguments on which the suggestions are based.

B-6. Reliability.—Quoting the Report:

> "All dynamic pile-driving formulas are subject to definite limitations and any dynamic pile-driving formula is nothing more than a yardstick to help the engineer secure reasonably safe and uniform results over the entire job."

NOTE.—This Report was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Messrs. G. G. Greulich, C. O. Emerson and D. O. Northrup, Harry J. Engel, and John D. Watson; October, 1941, by Messrs. Robert D. Chellis, Lazarus White, John G. Mason, Carlton S. Proctor, George Paaswell, and Abraham Woolf; November, 1941, by Messrs. Howard T. Evans, William G. Atwood, Donald M. Burmister, Wallace E. Belcher, Clement C. Williams, and D. P. Krynine; December, 1941, by Messrs. Trent R. Dames and William W. Moore, Maxwell M. Upson, Gregory P. Tschebotarioff, Robert F. Legget, and Jacob Feld; and January, 1942, by Messrs. Lewis C. Wilcoxen, H. A. Mohr, and A. E. Cummings.

[67] Dr. Ing., Lecturer, Harvard Univ., Cambridge, Mass.

[67a] Received by the Secretary December 22, 1941.

[68] "Dynamic Pile Driving Formulas," by A. E. Cummings, *Journal*, Boston Soc. of Civ. Engrs., January, 1940, Vol. XXVII, No. 1, p. 6.

If a manual recommends the use of one of several yardsticks, the reader is entitled to ask (*a*) which one of the yardsticks is the best, and (*b*) to what extent can he rely upon the results of the measurements with this yardstick.

These vital questions can be approached in the following manner: The defects of the pile formulas are either due to disregarding variable and vital factors ("Engineering News" formula), or they are due to inadequate evaluation of the influence of these factors on the effect of the blow of the hammer (general equation and its derivatives). The formulas of both groups share the defect that they disregard the energy transmission through the pile by elastic waves. The sign and the importance of the error due to these defects vary with the conditions. The degree of reliability of a formula can be measured by the range of scattering of the ratio between computed and real values about the statistical average.

In April, 1941, the Committee placed at the disposal of its members a valuable and diversified set of data,[69] assembled by G. G. Greulich, Assoc. M. Am. Soc. C. E., a member of the Committee. In Fig. 11 these data are shown in a logarithmic plot, the several formulas being identified by key letters defined on the drawing. For example: N = the "Engineering News" formula with the factor of safety eliminated; $M = N$ with the factor $\frac{P}{W}$ in the denominator; H = Hiley's formula as simplified by the Joint Committee, etc. Group (*a*) represents timber piles (single-acting hammer and cohesive soil), group (*b*) concrete piles (single-acting hammer and cohesive soil), and group (*c*) steel H-beam piles (various hammers and soil types). The equations that were used for computing the theoretical values are indicated by symbols, and the symbols are explained in the text accompanying the illustrations. (In Mr. Greulich's tabulation the general equation is called the Terzaghi equation. When the writer was compelled to learn this equation at college, it was then almost half a century old and had been used extensively. Hence, when introducing this equation into the paper cited by Mr. Greulich, he omitted the reference quite innocently. The equation appeared for the first time in J. F. Redtenbacher's "Prinzipien der Mechanik und des Maschinenbaues," published in 1859.) The value of 100% in Fig. 11 represents the measured test load, and the abscissas of the individual points represent the computed load expressed as percentages of the real load. All the points are confined within two vertical lines representing the values of 25% and 400%, respectively.

It is generally agreed that a pile foundation is not satisfactory unless the factor of safety of the individual piles with reference to the measured load at failure Q_1 is equal to, or greater than, two. When examining Fig. 11 one notices that the points representing the "Engineering News" formula (points on the first line written in each primary rectangle in Fig. 11) are grouped around a value of 150%. The formula is used with a theoretical factor of safety of 6. Therefore, the average real factor of safety for the group represented in Fig. 11 is $\frac{6}{1.5} = 4$. Since a real factor of safety of two is usually ample, the value 4 indicates that the application of the formula to the design

[69] *Proceedings*, Am. Soc. C. E., September, 1941, p. 1391.

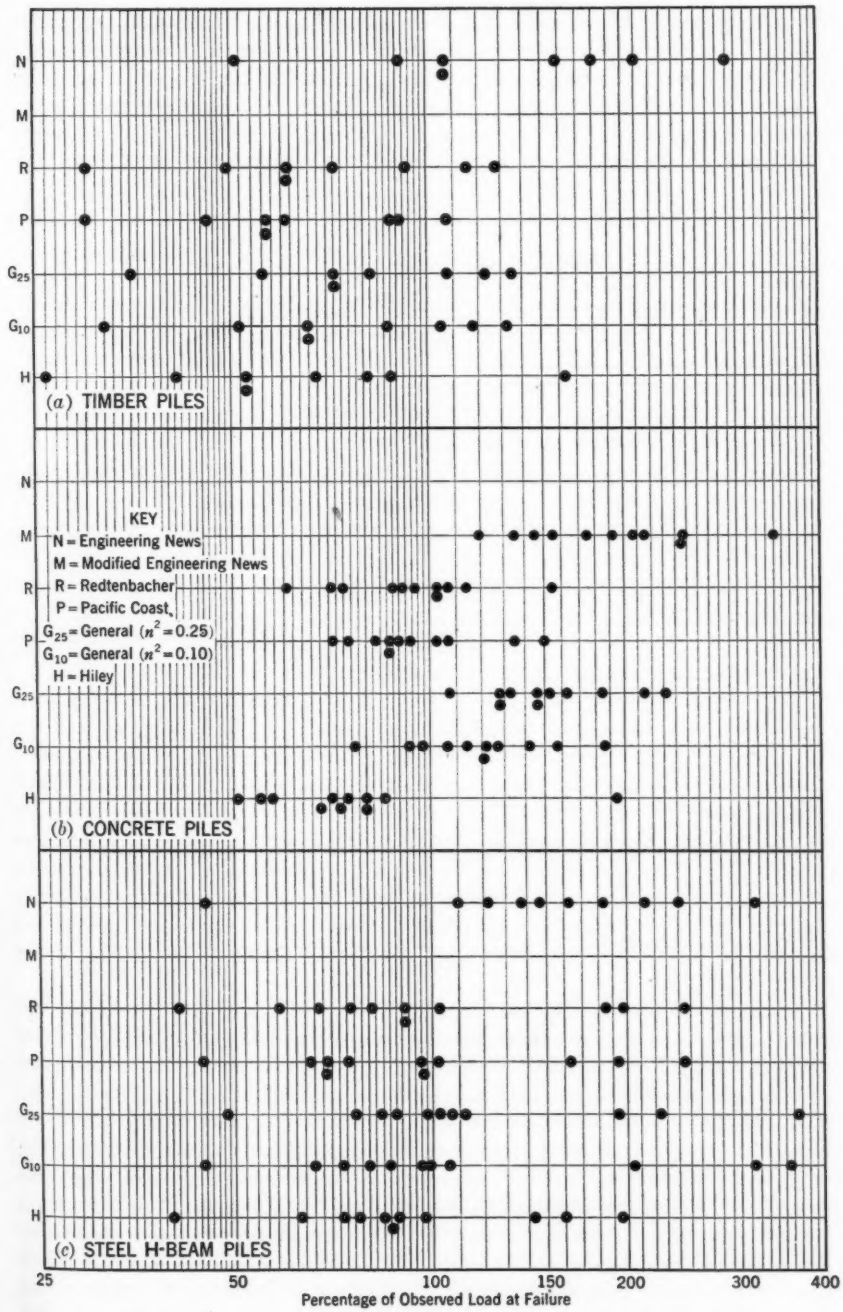

FIG. 11.—SCATTERING OF THE "RATIO" BETWEEN LOAD AT FAILURE COMPUTED BY MEANS OF DIFFERENT PILE FORMULAS AND THE OBSERVED LOAD AT FAILURE

of the foundations included in the group would have led on the average to driving twice as many piles as necessary. Nevertheless, the average real factor of safety of 4 is the smallest one compatible with the degree of accuracy of the formula. Otherwise, the number of piles under some of the foundations would be inadequate. This is demonstrated by the fact that the real factor of safety of the foundations represented by the points in the top line of Fig. 11(*c*) ranges between the extreme limits of $\frac{6}{0.45} = 13.3$, which is wasteful, and $\frac{6}{3.1} = 1.9$, which is inadequate.

To obtain a better approximation for the average real factor of safety, it would be necessary to repeat the investigation for a diversified group with several hundred members. However, the writer does not doubt that the value thus obtained would be closer to 5 than to 3. Otherwise, it would be inconceivable that the selection of the allowable load by means of the formula very seldom results in overloading the piles.

In spite of the waste of material and labor involved in an average factor of safety of approximately 4, an occasional failure is inevitable. This is demonstrated conclusively by Fig. 11. When assigning to each pile one sixth of the computed load at failure, the piles represented by the last point on the right-hand end of the first line in Fig. 11(*c*) carry their load with a real factor of safety of 1.9, which is inadequate; yet Fig. 11 represents not more than thirty cases. According to the laws of probability the real factor of safety occasionally must be considerably smaller than 1.9.

The preceding discussion leads to the following conclusions: Loading tests are justified economically if the cost of the tests is smaller than one half the cost of the piles, labor included, because, on an average, the saving amounts to about 50% of the cost of the foundation designed on the basis of the "Engineering News" formula. In addition to saving money the tests also eliminate the risk of overloading the piles. This risk always exists, when the formula is used. There is nothing to support the contention that judgment could assist in reducing the uncertainties revealed by the scattering of the values in Fig. 11. Whoever uses the formula is in exactly the same position as the man who tries his luck on a gambling machine. He is at the mercy of the laws of probability.

A superficial examination of the data shown in Fig. 11 produces the impression that the complicated equations (third to sixth line) give slightly better results than the "Engineering News" formula. However, on closer examination one finds that the superiority of the complicated equations is only apparent. By computing the standard deviation for the values shown in Fig. 11, R. B. Peck, Jun. Am. Soc. C. E. (in letters to the writer dated December 16 and 23, 1941), reports that the results obtained by means of the general equations are no better than those obtained by means of the "Engineering News" formula. For the time being there is no tangible evidence in favor of the contention that the scattering could be reduced by determining the items contained in Hiley's formula by field observations, but the act of making the observations would be a heavy burden on the engineer. If an engineer still believes in this

formula, he may experiment at his own risk, but he cannot challenge the preceding statement until his data include at least thirty cases, covering a considerable variety of conditions. In a discussion of pile formulas, only statistical arguments have any weight. In former years the writer has repeatedly expressed the opinion that the derivatives of the general equation may be more reliable that the "Engineering News" formula, and he shared this opinion with many other engineers. The data shown in Fig. 11 demonstrate that this opinion was unwarranted, and Mr. Cummings has shown[68] that it could not be otherwise.

The preceding interpretation of the data represented by Fig. 11 must be considered a vital supplement to those passages in the Manual that deal with the "yardstick," lest some readers may believe that the "yardstick" is at least approximately "one yard long." For this reason, the writer suggests that the Committee incorporate into the Manual either Fig. 11, or a similar figure containing more points, and that the sentence quoted at the beginning of this discussion be replaced by the following:

> The degree of reliability of the different pile formulas is indicated by Fig. 11. It shows the scattering of the ratio between the bearing capacity computed by means of seven different equations and the real value. By increasing the number of observations the extreme limits of the range of scattering must, of necessity, increase, whereas the average scattering may either increase or decrease. There is no evidence that the results obtained by means of the derivatives of the general equation are more reliable than those furnished by the "Engineering News" formula.
>
> The average real factor of safety of pile foundations designed on the basis of the "Engineering News" formula with a theoretical factor of safety of 6 is most likely about 4. The real values for individual foundations range between about 2 and 13, but in exceptional cases the real value can be smaller than 2, which is inadequate, or greater than 13, which is utterly wasteful. These data lead to the following conclusions: When designing a great number of pile foundations on the basis of the "Engineering News" formula, the total number of piles is roughly equal to twice the number that would suffice to support the loads safely. Nevertheless, on a few jobs the number of piles would be inadequate. Hence, the use of the formula on the design of pile foundations is unsound on both economical and technical grounds. An exception to this statement can be made only if the cost of the load tests is greater than about one half of the cost of the pile foundation.

The preceding statements are based on data which the Committee has submitted to its members for consideration. If the Manual fails to include Fig. 11 or a fully equivalent text treatment, it also fails to give its readers an unbiased conception of the merits and faults of the pile formula.

B-7. Limitation of Driving.—This section should contain at least a few words concerning the condition that the driving of timber piles should be discontinued before the pile breaks under the impact of the hammer. Fig. 11 shows that the theoretical value of the pressure exerted by the blow of the hammer on the pile ranges between about 25% and 400% of the real value; yet point-bearing piles are quite often loaded to within one third or one fourth of the ultimate compressive strength of the timber. Therefore, the pile formula as such does not permit a decision as to whether or not the danger

point is approached during the process of driving. When the representatives of different concrete pile systems in Holland became interested in the condition of foundations on timber piles, it was found that a surprisingly large number of timber piles were broken; yet the Dutch pile formulas are no worse than the others. To avoid the danger of breaking, one could drive a few test piles until they break and select the allowable minimum penetration for the last blows on the basis of the pile-driving record for the last few feet. In connection with this selection, a pile formula (such as the "Engineering News" formula) could be used, not for the purpose of computing the pressure on the pile immediately prior to failure, but for estimating roughly the ratio between actual load and load at failure for different penetrations.

B-8. Static Formulas; Reasons for Use.—The Committee states: "Most of the available static formulas are based on the classical earth pressure theories and some of them are rather elaborate." In this section distinction should be made between scientific formulas, involving certain assumptions, and the purely empirical formula for friction piles,

$$R = \frac{A_s f}{3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (64)$$

in which R = allowable load, A_s = area of the friction surface, and f = the value of friction per unit of area. Eq. 64 does not involve any assumption other than that f is known.

All the scientific theories covered by the bibliography in Paragraph *B*-11 are based on the assumption that a cylindrical part of the soil has been replaced, through some act of magic, by a pile. Before the load is applied the state of stress in the soil should be identical with what it was before the pile was created. In reality, the pile must be driven or jetted into the ground. Either process produces a profound change in the state of stress in the soil and in some cases alters the physical properties of the soil which surrounds the pile. The nature of these changes is very different for various soils and diverse methods of installing the pile in the soil. When attempting to modify the theory in such a manner that these changes are taken into consideration,[70] one realizes that the compressibility of the soil is a factor of decisive importance and the labor required for evaluating the constants in the equations becomes prohibitive, if it is possible in any case. The theories that disregard the aforementioned changes (Messrs. O. Stern, Heinrich Dörr, John H. Griffith, and others) are no more reliable than the pile formulas. Almost all the successful computations of the bearing capacity of piles on the basis of scientific static theories were made after the load test was finished and not before. According to all the theories, the friction per unit of area of the surface of piles in homogenous clay should be equal to a constant plus a quantity that increases in simple proportion to depth. This conclusion is conspicuously incompatible with experience. For these reasons the writer suggests the exclusion of the scientific formulas from consideration in the Manual and the elimination of items (4), (5), and (6) from the bibliography in Paragraph *B*-11.

[70] "Erdbaumechanik auf bodenphysikalischer Grundlage," by K. Terzaghi, Leipzig, 1925.

The use of Eq. 64 requires that the value f be known. In this connection Paragraph *A*-14 suggests that the value f should be "determined carefully in each case, bearing in mind that it may increase or decrease with the depth." The writer does not know how this could be done, and he wonders whether anybody else knows. However, as a supplement to this advice, Paragraph *A*-14 presents a list of measured values of f (see Table 1). Since these values are familiar to the writer, he knows that some of them were determined by loading but most of them by pulling tests. However, the reader does not know it, because Table 1 does not contain any references. Furthermore, the table does not contain any data concerning the time that elapsed between driving and testing the pile. The following incident shows that the omissions are incompatible with the practical purpose of the table:

The writer designed a pile foundation on friction piles near the coast of Scotland. To a depth of at least 150 ft the soil consisted of a soft brown clay with streaks of silt. The plastic limit ranged between 20% and 22%, the liquid limit between 37% and 45% (in exceptional cases, 50%), and the natural water content, at every depth, down to 120 ft, was slightly greater or less than the liquid limit. When driving piles (12-in. by 12-in. to 14-in. by 14-in. composite piles with parallel sides, 70 ft long) into this clay, the clay temporarily and locally passed into a viscous liquid state. This was demonstrated by the following observations: When a pile was being driven, it rose after each blow, including the last one, through a distance between 4 in. and 6 in., although the permanent penetration produced by each blow was frequently smaller than $\frac{1}{2}$ in. One of the piles followed the hammer after the last blow and rose 15 ft. To eliminate this nuisance, which seriously interfered with the pile-driving operations, it was necessary to provide the rigs with an attachment that prevented the piles from rising after the blows. All the piles had to be redriven at least once, because while the last piles in a cluster were being driven the first ones rose from 1 to 3 ft. About twenty of the piles rose from 5 to 10 ft. When the first pile in a new cluster was being driven, about half an hour after the last pile in the preceding cluster had been driven, the latter rose slowly about 20 ft, although it was located at a distance of about 40 ft from the new pile. The piles were driven with 2-ton to 3-ton drop hammers, with 25 to 30 blows per min. Double-acting steam hammers, delivering 220 blows per min, were found to be utterly unsatisfactory.

The writer proposed to establish the foundation for the buildings and for heavy, vibrating machinery on friction piles 70 ft long and to assign a load of 15 tons to each pile. The selection of the load per pile had to be made in a hurry before any test piles were driven. It was based on the writer's previous experience with pile foundations on what appeared to be similar clay deposits. On the basis of a load of 15 tons per pile, the foundation required about 3,500 piles.

To determine whether a load of 15 tons per pile is safe, test piles were driven to different depths, ranging between 27 and 88 ft. Five loading tests and six pulling tests were performed by means of the bootstrap method, and four load tests were made by loading with pig iron. The latter tests were combined with an investigation of the influence of intense vibrations on the settlement of individual piles.

The behavior of the test piles during driving indicated quite plainly that the point resistance is negligible. If the piles can carry any load, they carry that load by skin friction. The very first test demonstrated that the skin friction is practically equal to zero immediately after driving and that it increases with time according to some unknown law. To become acquainted with this law it was decided to repeat the loading test on each pile in time intervals of several days. Each test consisted of increasing the load on the test pile gradually until the penetration became greater than 2 in. and then removing the load completely. The elastic rebound was always of the order of magnitude of $\frac{1}{2}$ in.

The first tests also showed very emphatically that the skin friction which resisted an upward movement of a pile was much smaller than the resistance against penetration. The ratio between these two values, for adjacent piles, was practically independent of time. For different locations it ranged between the limits of 2.1 and 2.2 (short piles) and 2.4 and 2.6 (long piles). To make (for instance) a bootstrap test on a 12-in. by 12-in. pile with a penetration of 88 ft, it was necessary to drive the anchor piles (14 in. by 14 in.) to a depth of 102 ft. Nevertheless, the anchor piles rose almost one inch when the load on the middle pile approached the value of the load at failure.

The bed of clay was statistically homogenous from the base of the shallow top layer, consisting of peat, down to a considerable depth below the points of the test piles. Nevertheless, the influence of time on the skin friction depended to a large extent on the length of the piles. For the three piles that were shorter than about 40 ft, the load at failure six days after driving was slightly smaller than the corresponding load two days after driving. On the other hand, the bearing capacity of all the other piles with a length between 65 ft and 90 ft increased at a decreasing rate and became practically constant approximately four weeks after driving. In Fig. 12 the upper curve represents the results of a series of load tests on an 85-ft pile and the lower curve those of a series of pulling tests on a 70-ft pile. After each load application (represented by a plotted point), the load was removed completely before a new application. In the lower curve each plotted point represents the skin friction at failure upon each re-application of the pulling force.

There is no reason for assuming that the clay deposit to which the preceding data refer should be unique. There must be many similar deposits in most parts of the world, including the United States. There is no reason, either, to assume that the results of the tests in Scotland should furnish more than qualitative information regarding the skin friction on piles in other clays. In some clays the skin friction does not materially increase after the piles have been in the ground one week, and in others the increase of the skin friction with time may be greater than that shown in Fig. 12. In some clays the force required to drive the pile into the clay is almost equal to that required to pull it. In the Scotch clay it was about 2.5 times greater, and in other clays it may be still greater. No one seems to know anything definite about it. The influence of the length of the pile on the time effect was also unknown to the writer until he made the foregoing tests. Fig. 12, which illustrates

these tests, leads into almost unexplored territory. Therefore, the writer suggests that the Committee digest the contents of its files, assemble whatever can be learned about skin friction in tables with descriptive text, and publish the tables in an Appendix to the Manual. In 1929, when serving as a chair-

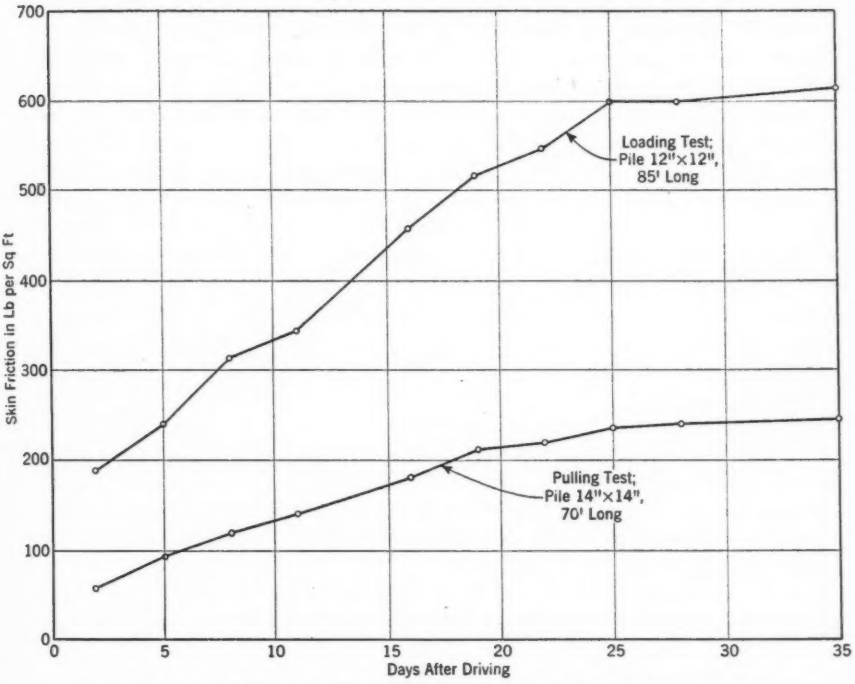

FIG. 12.—TESTS ON SQUARE TIMBER PILES WITH PARALLEL SIDES IN A SILTY CLAY NEAR THE COAST OF SCOTLAND

man on a subcommittee, the writer submitted to the Committee a tentative report containing a chapter entitled "Effect of Time on the Bearing Capacity of Individual Piles." The writer hoped that this chapter would serve as a nucleus for gathering supplementary data concerning this important topic; yet the Report contains less than what was known in 1929.

The digest of the contents of the files of the Committee has not yet been started. Therefore, the writer proposes to introduce the empirical Eq. 64 from Paragraph *A*-14 into Paragraph *B*-8, together with the following comments:

> The value f in this equation depends on the nature of the soil, on the time that elapsed since driving the pile, and several other factors whose influence cannot yet be determined by laboratory tests. The skin friction which resists a downward movement of the pile can be very much greater than the frictional resistance against pulling, but the two values can also be almost equal. Preliminary information regarding the influence of all these factors on the value f can be obtained only on the basis of

case records covering a great number of different conditions. Conclusive information can be secured only by loading tests, unless similar tests have previously been made in the vicinity of the site.

B-9.—The Report states "The determination of the physical properties of the surrounding soil is a difficult problem as the driving of the pile often produces changes in the physical characteristics of the soil." This statement can be replaced safely by "* * * is still an unsolved problem," and the remainder of the paragraph can be omitted.

B-10.—Quoting the Report: "The use of either a dynamic or a static formula must be accompanied by a considerable amount of experience and good judgment." The writer cannot understand where judgment enters. The scientific static formula should not be used in any case. When using the empirical formula, Eq. 64, one needs a table containing a great number of friction values, referring to very different geological conditions, with adequate data concerning the other circumstances associated with the tests. The degree of reliability of the estimate depends primarily on the number and quality of the data in the table. Fig. 11 shows that the author of the "Engineering News" formula exercised excellent judgment when selecting a factor of safety equal to 6. However, this was the only instance in the history of this formula when judgment was required.

In Paragraph *B*-6 the Report states that "The use of a complicated formula is not recommended * * *," yet the second part of Paragraph *B*-10 gives instructions for determining the various constants in the most complicated of the existing formulas. Since this is contradictory to Paragraph *B*-6, the entire paragraph should be eliminated.

B-11. Bibliography for Report B, on Pile Formulas.—Items (4) to (6) of the bibliography should be omitted for the reasons given in the writer's comments on Paragraph *B*-8.

B-15.—This topic requires a less categorial treatment, lest some readers may believe that the rule stated in this paragraph is applicable under any circumstances. In addition, the meaning of the term "load at failure" should be defined. According to prevalent usage, the term indicates "the load that produced an increase in settlement disproportional to the increase in pile load" (item (2), Paragraph *B*-11). Figs. 13 and 14 show the results of two different load tests (*a*) and (*b*) on wood piles 35 ft long. In Fig. 14 each of the two load-settlement diagrams shown in Fig. 13 has been plotted to a different settlement scale. When applying the foregoing rule without taking the first steep descent of the curves into consideration, one obtains for the pile (*a*) from Figs. 13(*a*) and 14(*a*) the values 80 and 95 tons, respectively. They are at least of the same order of magnitude; but what is the failure load of the pile (*b*) in Figs. 13(*b*) and 14(*b*) according to the foregoing definition? The writer would not even dare to make a guess. The Manual should specify somewhere that the failure load is not reached unless the penetration of the pile is at least equal to 10% of the diameter of the tip of the pile. At smaller penetrations, not more than a fraction of the ultimate point resistance of the pile has been mobilized.

The empirical rule that constitutes the body of Paragraph *B*-15 does not contain anything about the time that should have elapsed between driving the pile and testing it. When dealing with friction piles such as those whose properties are illustrated by Fig. 12, the time can be a factor of major im-

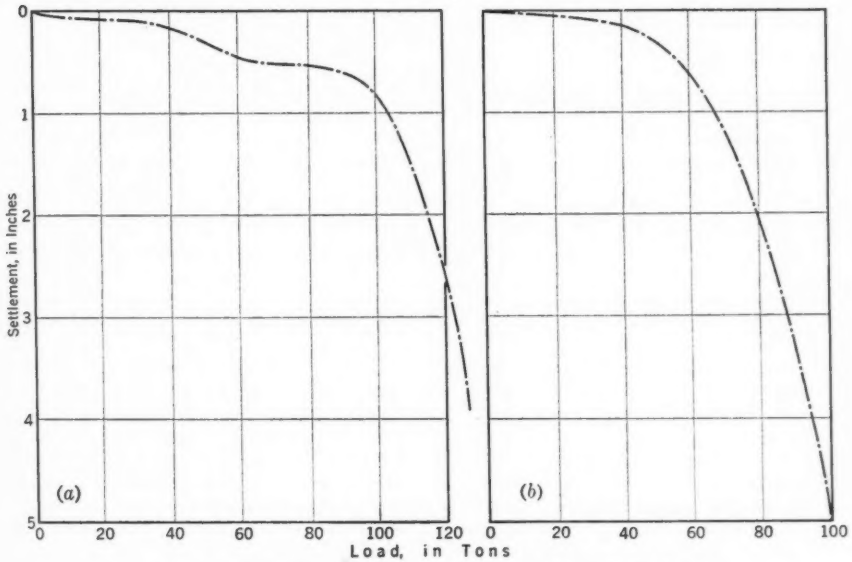

FIG. 13.—LOAD SETTLEMENT CURVES FOR WOOD PILES 35 FT LONG

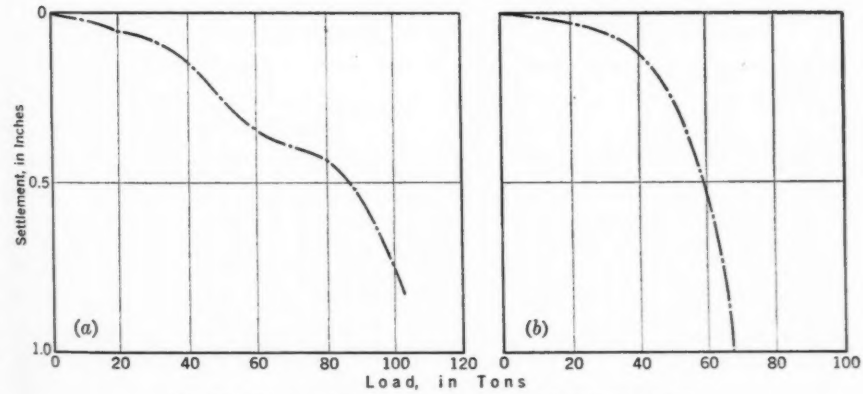

FIG. 14.—SAME AS FIG. 13, WITH DIFFERENT VERTICAL SCALES

portance. No distinction is made between the allowable load on piles subjected to a fairly constant load and others that must support very variable loads. Friction piles are likely to be sensitive to periodic load variations. Finally, the local geological conditions must be taken into consideration. If the subsoil is fairly uniform, the information obtained from a few pile tests con-

cerning the average bearing capacity is fairly reliable. On the other hand, if the soil conditions change from drill hole to drill hole, one must be careful. For these reasons the writer suggests replacing Paragraph *B*-15 by the following:

> The design of a pile foundation involves two entirely different and independent steps. The first one consists in determining the depth or the stratum to which the piles should be driven, and the second one consists in computing the number of piles required to carry the load. The depth of penetration is determined by the geological conditions or by the condition that the settlement of the entire foundation should be compatible with the character and the purpose of the superimposed structure. During this investigation the bearing capacity of the individual pile does not even enter into the picture. The procedures involved in the first step have been discussed in another part of the Manual.
>
> The second step consists in computing the number of piles by dividing the total load by the allowable load per pile. The allowable load is equal to a certain fraction of the greatest load that an individual pile could carry. In many cities of the United States the following simple rule has given consistently satisfactory results: The test pile should be tested under twice the load Q that will be assigned to each pile. When loaded with 2 Q, the pile should not yet show any signs of failure. The difference between the settlement under the load 1.5 Q and the rebound due to the removal of the load 1.5 Q should not exceed 0.01 in. per ton of load.
>
> In the cities in which this rule has acquired its excellent reputation the points of most of the piles are driven into a firm stratum. Therefore it is by no means certain that this rule could be applied safely to typical friction piles. As a matter of fact, in cities where the buildings rest on typical friction piles, quite different rules for determining the allowable load have grown out of experience, and these rules are different in different cities, depending on the prevalent type of soil. Hence, if the bearing capacity of the piles depends chiefly or entirely on skin friction, the appropriate selection of the allowable load requires mature judgment and considerable experience; the degree of uniformity of the subsoil, the number of loading tests, the characteristics of the load-settlement curve, possible time effects, and several other factors must be considered.

B-14. Method of Loading.—In this paragraph the Committee recommends that the anchor piles in the bootstrap tests be located at a distance of not less than 10 ft from the test pile. In the tests illustrated by Fig. 12 the distance between the test piles and the anchor piles was 5 ft, center to center. The results of the bootstrap tests have been checked by means of four direct load tests and were found to be more than sufficiently reliable. Some engineers consider a distance of 5 ft inadequate. However, the writer has never seen any test data to confirm this opinion, and he believes, on theoretical grounds, that a distance of 5 ft is ample, provided that the diameter of the piles does not exceed about 1 ft. At the surface of the anchor piles the shearing stresses are smaller than the shearing strength of the soil. Otherwise, the anchor piles would be lifted out of the ground. The shearing stresses on vertical, cylindrical sections through the soil decrease approximately in inverse proportion to the distance from the center line of the anchor pile. Hence, at a 5-ft spacing and a pile diameter of 1 ft, the shearing stresses in the soil on vertical sections halfway between the load and the anchor pile are smaller than one fifth of the skin friction. There is no reason for assuming that these

insignificant stresses should have a considerable influence on the load at failure for the middle pile. Whenever the writer made bootstrap tests, he always drove pegs into the ground along the line connecting the two anchor piles, and he never noticed more than a slight movement of these pegs beyond a distance of 1 ft from the anchor pile. Therefore, he suggests replacing the condition mentioned at the beginning of this discussion by the condition that the distance between the load and the anchor piles should not be less than five times the diameter of the anchor piles.

Summary of Recommendations.—The mathematics in Report *A* are incompatible with the contents and the spirit of the only scientifically sound analysis of the problem that has been published to this time. Therefore, Report *A* should either be eliminated or rewritten and published in an Appendix.

Paragraph *B*-6, on the reliability of pile formulas, fails to convey quantitative information to the reader, although the need for such information is recognized by every one. The missing information has been presented by the writer both in a graph and in supporting descriptive matter. It has been obtained by applying the elementary rules of statistics to a group of 30 tests. The results of the analysis remove whatever doubts may exist regarding the conditions to be satisfied in order to justify, on economic grounds, the application of pile formulas to the design of pile foundations. If the Manual fails to include these or fully equivalent data, it also fails to provide the reader with unbiased information regarding this controversial topic.

The only objection that can be raised against the data supplied by the writer is that a group subject to statistical investigation should include more than 30 tests. This objection could easily be met by repeating the investigation on a group of 100 or more tests. The files of the Committee contain more than ample material; yet it is the writer's contention that the results would not alter the order of magnitude of his figures.

Paragraph *B*-15 contains a rule for determining the allowable load on piles without mentioning the limits for its safe application. Suggestions for revising this paragraph have been made.

These are the major recommendations. For minor recommendations the main body of this discussion may be consulted.

The writer wishes to congratulate the Committee on having practically finished the arduous task of compiling the Manual after twelve years of sincere efforts. He expresses the hope that the Committee may now embark on the less academic but far more important task of digesting the contents of its voluminous files containing case records. This could be accomplished in one or two years by an intelligent beginner under the guidance of an experienced engineer who knows "what he is driving at." In the field of piles and pile foundations, generalities represent a "body" without substance and the theories are somewhat "anemic."

RALPH B. PECK,[71] JUN. AM. SOC. C. E.[71a]—Report *A* carries the implication that pile-driving formulas give results that have some relationship to the

[71] Asst. Subway Engr., Dept. of Subways and Superhighways, City of Chicago, Chicago, Ill.
[71a] Received by the Secretary December 23, 1941.

ultimate bearing capacity of piles. The validity of some or any of these formulas can be determined only by comparison of ultimate loads found by loading tests and by the formulas.

For those among engineers who are interested in statistics, the following reasoning is submitted. On the basis of the data in Table 2,[72] it can be demonstrated by a purely statistical approach that the chances of guessing the bearing capacity of a pile are better than of computing it by a pile-driving formula. Table 14 gives the necessary statistical parameters computed from Table 2. Values in the latter table were eliminated for nine tests in which the solution of Eq. 23 was not given.

Assume that a new and very simple pile formula is advanced. It is merely the statement that the most probable bearing capacity of every pile is 91 tons. (This value happens to be the geometric mean of the tested ultimate bearing capacities taken from Table 2.) The standard deviation of the individual bearing capacities about this value is 1.55, which is less than the standard deviation determined for any of the pile-driving formulas. To determine the ultimate bearing capacity of a pile, the following procedure then would be justified: Take 100 poker chips and label them with numbers so as to form a geometrically normal array having a mean value of 91 tons and a standard deviation of 1.55. Mix the poker chips and select one. The value written on the chip will be the bearing capacity of the pile. Furthermore, the value from the chip, according to the basis of "cold statistics," will be nearer to the true bearing capacity more frequently than a value obtained by use of any of the pile-driving formulas represented in Table 14.

TABLE 14.—STATISTICAL PARAMETERS

Formula		Geometric mean[a]	Standard deviation	Standard deviation of mean itself
Name	Eq.			
Data (load tests)	..	100%	1.55	1.10
Hiley	22	78%	1.62	1.11
Engineering News	23	166%	1.88	1.15
Redtenbacher	19	88%	1.71	1.12
Pacific Coast	25	85%	1.69	1.12
General ($n^2 = 0.25$)	26	105%	1.69	1.12
General ($n^2 = 0.1$)	26	97%	1.75	1.13

[a] Percentage of test load at failure.

The statistical study indicates that the use of a pile-driving formula is merely a somewhat inferior method of permitting the laws of chance to operate in the determination of pile capacity. For any individual pile, the computed bearing capacity may be on the unsafe side or may be ultraconservative, without any previous way of determining which will be the case. This might be expected on the basis of the work of Mr. Cummings,[68] who has shown that all of the common formulas are illogical, and on the basis of the entirely unknown (and perhaps nonexistent) relation between dynamic and static pile resistance.

ARTHUR CASAGRANDE,[73] ASSOC. M. AM. SOC. C. E.[73a]—The question of "pile formulas" has without doubt been the most controversial issue in the field of civil engineering for a hundred years. Judging from the work of the Com-

[72] *Proceedings*, Am. Soc. C. E., September, 1941, p. 1393.

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[73a] Received by the Secretary January 13, 1942.

mittee, and from other evidence, opinions on this subject are today more divided than ever.

The Fallacy of Energy Loss Due to Elastic Compression of Pile.—In Report *A*, Eq. 9 is recommended as the most satisfactory pile formula devised to date. As shown for the first time by Mr. Cummings,[25] all formulas that deduct a Newtonian impact loss and in addition a loss due to the elastic compression of the pile, soil, etc., are inherently wrong, since "these are already included in the Newtonian term." In other words, the error in the so-called complete formula is a question of superposition.

The writer agrees fully that the elastic compression of pile and soil must not be deducted as energy loss in addition to an energy loss derived from the Newtonian theory. In fact, the writer believes that the energy temporarily stored up by elastic compression should not be considered as a loss "already included in the Newtonian term," but that it is no energy loss at all so far as testing the dynamic penetration resistance of the pile is concerned. On the contrary, the writer considers this elastic compression a measure of the energy with which the soil is being tested. Perhaps the following illustration may assist in understanding the rôle which elastic compression plays during the pile-driving operation.

Consider the time-honored analogy of the impact of billiard balls. A moving ball striking a ball at rest will cause a temporary elastic compression of both balls at the moment of impact. It is really this elastic compression that temporarily stores up and transfers energy. If the balls were perfectly elastic, all the energy stored up in deforming the balls would be recovered in the form of kinetic energy of the second ball. The more the stress-strain characteristics of the material deviate from a perfectly elastic material, the greater is the proportion of energy that is lost in permanent deformation. It is this loss which is reflected in the coefficient of restitution.

Consider now a row of billiard balls touching each other. Another ball is allowed to strike against one end of this row, with the result that this ball will remain at rest in the position of impact, while the last ball at the other end of the row will shoot away. In this case, the elastic deformation, which represents the energy transferred from the moving ball, is transmitted by elastic compression from one ball to the other. When this "compression wave" reaches the end of the row, it kicks the last ball off with an energy equal to the energy which the first ball transmitted to the row, minus energy losses due to non-elastic deformation.

Now, assume that the last ball of the row is in contact with a wall of plastic clay. The result would be that the ball would force itself into the clay, creating as much permanent deformation as the available energy would permit; and then the ball would stop. On the other hand, if the last ball were in contact with a perfectly elastic wall (a very large mass in relation to the mass of the ball), the wall would temporarily be deformed, and recovery of this deformation would return the energy to the last ball and back through the row, and eventually the first ball would rebound in the direction from which it came. In the first case, with plastic reaction, all energy was used up in permanent deformation of the

[25] *Journal*, Boston Soc. of Civ. Engrs., January, 1940.

clay; but before this permanent deformation could take place, the energy was temporarily stored up in the form of elastic compression of the balls. In the second case, with a perfectly elastic reaction, the energy was imparted to the reaction (that is, the reaction was "tested") but, being a perfectly elastic material, the deformation was recovered and thereby the energy was returned to the row of balls, eventually shooting back the first ball.

These simple experiments illustrate sufficiently that it is the elastic compression of the pile which transmits the energy to the point and produces permanent penetration.

Assuming the pile to be elastic, the energy will either cause a permanent impression in the soil at the tip, or, if the reaction is almost elastic (hard rock), it will return to the pile. In the latter case, it is important to note that the energy has also "tested" the material beneath the tip of the pile. Thus, to measure the "temporary" (that is, elastic) compression of the pile and soil and deduct the corresponding energy as "loss" is fundamentally wrong. It would be more correct to consider the elastic compression of the pile as a measure of the force with which the soil is "tested."

Consider the extreme case of a very slow application of the energy to the pile—that is, a static load test. If the elastic coefficients of pile and soil, and the manner in which the force is transferred to the soil, are known, the elastic compression of the two (which can be determined after removal of the load) would permit computation of the load with which the pile was tested.

The other extreme would be such a rapid transfer of energy to the pile that the stress concentration would far exceed the elastic limits and even the strength of the material; as, for example, by shooting a small cannon ball against the butt of the pile. Although, in this case, just as much energy may be imparted as by one blow of a pile hammer, or by a static load test, the energy transfer is concentrated in so small a part of the pile, and in so short a time interval, that it will cause local destruction of the material, while at the tip of the pile practically no impact is felt.

If the elastic compression of the pile and soil at the moment of impact of the cannon ball could be measured, it would be found to be extremely small, and the force with which the pile is being tested against quick penetration into the soil is correspondingly small.

Although, in a sense, the elastic compression is a measure of the force with which the pile is tested against dynamic penetration into the soil, the problem is so complicated that there is little hope that direct measurement of the elastic compression will permit quantitative conclusions regarding the dynamic penetration resistance of the pile. To illustrate the difficulties, the following measurements are cited:

In the construction of a steel pipe pile foundation, the writer conducted measurements of the elastic compression by means of a drum which was rotated with uniform speed once with every blow of the hammer. A pencil attached to the pile was pressing against the paper on the drum, thereby recording with considerable accuracy the actual movement of the pile. Three typical curves are shown in Fig. 15. In the upper curve the permanent penetration was 1.13 in., with practically no elastic rebound of the pile. Fig. 15(*b*) shows a

permanent penetration of 0.44 in. and an elastic rebound of 0.35 in. The lower curve shows a permanent penetration of only 0.1 in., with an elastic rebound of 0.68 in.

The energy per blow was the same for all three observations. In these, as well as in many other observations, it was found that the elastic rebound in-

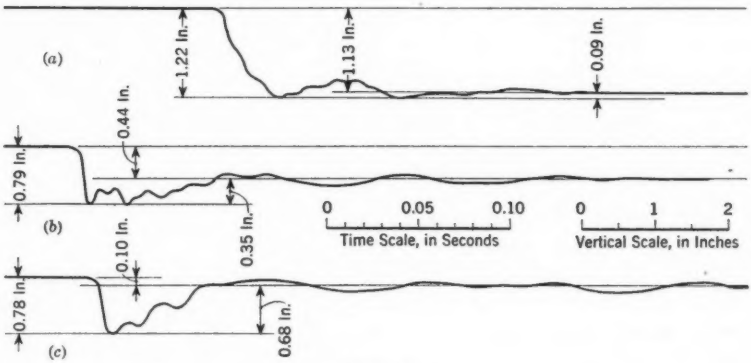

FIG. 15

creased with decreasing permanent penetration. This is not surprising if one considers the possible extremes—perfectly elastic reaction (hard rock), in which case the entire energy is used up for elastic compression of the pile and the underlying medium, which compression is recovered; or perfectly plastic reaction, in which case the elastic compression of the pile is recovered mainly in downward direction; that is, producing penetration.

The period of the longitudinal vibrations in the pile is of the order of 0.02 sec, whereas the superimposed vibrations of the ground, which are also visible in Fig. 15, are about five times slower.

In Fig. 15(*a*), the soil is tested by energy that was temporarily stored up in the form of elastic compression of the pile, but which cannot be derived from the curve. In Fig. 15(*b*) there is an elastic rebound of 0.35 in. Some of this is elastic compression of the soil which should be deducted from the elastic compression of the pile. On the other hand, a part of the elastic compression of the pile is recovered in downward direction and does not appear in the measured rebound. In Fig. 15(*c*) almost the entire downward movement is recovered in the rebound. Probably more than one half of this value is elastic compression of the material underlying the lower end of the pile. Since its value is unknown, the actual energy stored up elastically in the pile cannot be derived from this curve.

Simple Formulas Vs. Complicated Formulas.—In 1940, Mr. Cummings suggested in a lecture that the "Engineering News" formula may be preferable to the more complicated formulas, particularly to those formulas containing the elastic compression of the pile, etc.

The writer concurs in this opinion, since there is no doubt that the complicated formulas inspire more confidence. As an illustration, the following

formula is cited:

$$R = \frac{W\,h}{10\,s} \qquad (65)$$

which is derived from the relationship $R\,s = W\,h$, and in which the reduction factor (10) was found empirically by comparison of numerous load tests and driving characteristics. In this formula R is the allowable load, $W\,h$ the energy delivered per blow, and s the penetration, all quantities to be in a consistent system of dimensions. When the penetration per blow is less than 0.1 in., the value of s shall be taken equal to 0.1 in. Thus, for hard driving, the formula is reduced to $R = W\,h''$, in which h'' is a dimensionless quantity numerically equal to h measured in inches.

Eq. 65 is an empirical expression stating that by using only 10% of the total energy per blow, a safe load may be obtained; also that no less a penetration than 0.1 in. shall be used. This simple formula gives results that are as satisfactory or unsatisfactory as those of any other pile formula. In fact, the writer has found, in many cases, a more satisfactory agreement with the allowable loads as determined from load tests than any other formula. However, it is not the writer's intention to recommend Eq. 65, except for the specific purpose that readers who have collected pile driving and load test data may use this formula, as well as the "Engineering News" formula and more complicated ones, to convince themselves that complicated formulas have definitely no advantage over the simplest types.

Empirical Rules for Bearing Capacity of Piles.—The writer realizes that pile loading tests cannot be made a general requirement and that, although on large projects they should be used and pay for themselves, they are often too expensive for small projects. Therefore the problem is to find a more reliable basis than formulas for designing and constructing pile foundations for small projects, and a less expensive method than loading tests. Such a method can be developed for cities or other limited areas not having a great number of different soil strata. The best-known example of this "empirical method" is the City of Shanghai, China, where, on the basis of systematic load tests, a series of empirical rules was derived for the allowable loads for various types and lengths of piles, based on allowable friction values of the embedded area. There are many cities in the United States in which a comprehensive study of available load tests and of soil data would permit establishing empirical rules, which would definitely be superior to any formula.

Application of such empirical rules requires borings for every project, unless the soil conditions are sufficiently uniform to permit extrapolation from existing boring records on adjacent areas. Some engineers consider compulsory borings for small projects just as objectionable as pile loading tests. To this the writer replies, first, that a few exploratory borings are much cheaper than pile loading tests; and, second, that the subsoil conditions should be known fairly accurately even for small projects, since even the smallest buildings may get into serious trouble if designed and built without sufficient knowledge of the subsoil conditions.

If such empirical rules are not available, or applicable, one can utilize other empirical knowledge instead of formulas. In this connection, it seems pertinent to remember that there are a number of foundation engineers, in whose experience and judgment the writer has full confidence, who make their decisions on pile foundations on the basis of a detailed inquiry into the subsoil conditions and definitely without the use of pile formulas either in the design or the specifications. The writer has also observed that confidence in pile formulas is often the result of experience derived with one type of pile only, usually wood piles. Realization that something must be radically wrong with formulas comes when such engineers are suddenly faced with the problem of using an entirely different type of pile.

It would be very desirable if foundation engineers who have developed methods of approach that eliminate the necessity of using pile formulas would make their experience available to the profession. The writer would like to outline briefly his own approach to such problems:

First, the writer insists on a thorough subsoil exploration by means of continuous sampling of the pertinent strata, using 2-in. diameter thin-walled steel tubes. This method was originally developed in 1936 by Henry A. Mohr, M. Am. Soc. C. E., for the writer's requirements, and subsequently improved by M. J. Hvorslev, Assoc. M. Am. Soc. C. E. In addition, it is required that the resistance to penetration of the sampling tubes be measured. All samples are submitted to a visual and manual inspection in the laboratory and to such tests as the determination of the natural water content, the Atterberg limits, and the unconfined compressive strength.

This information is then digested and used as a basis for determining the type of foundation required to support the proposed structure safely. Special attention is paid to the presence of such soil strata beneath the foundation or the points of the piles, if piles must be used, which have objectionable compressibility characteristics.

The next step, if a pile foundation is chosen, is the selection of the type of pile and its allowable load. At this point, and for the remainder of this discussion, it will be assumed that the safe allowable load per pile bears no relation to the behavior of the entire foundation and the overlying structure. Serious settlements may, and often do, occur in spite of the fact that every single pile in the foundation carries a safe load, that is a load which would have to be greatly exceeded before the pile would move in relation to the immediately surrounding soil. In fact, the entire chapter on pile formulas is only intended to treat the question of determining, for specifications and in the field, the resistance to which a pile should be driven. The question of designing a foundation so as to prevent detrimental settlements is treated in other chapters of the proposed Manual, and is definitely beyond the scope of the chapter on pile formulas as well as of this discussion.

Usually the allowable load is already fixed by empirical rules contained in building codes. These values may be exceeded only on the basis of reliable load tests which automatically yield the necessary information on the final resistance, or embedment, to which the piles should be driven.

In the absence of load tests, pile formulas are supposed to determine the final resistance to which piles shall be driven. Unfortunately there are no provisions in these formulas to assure safety against overdriving; and it is well known that efforts to satisfy the "Engineering News" formula for drop hammers often result in destruction of wood piles.

Since pile formulas are basically incapable of yielding the desired information, and since they do not contain any provisions to prevent overdriving of piles, the writer has found it preferable to use the following empirical rule:

> "A pile driven to the maximum permissible resistance that will not harm the pile can be loaded safely to the maximum allowable loads permitted in building codes."

In most cases considerably less energy would be sufficient to attain the desired bearing capacity, and for friction piles driven into soft clay, this rule would cause the piles to be driven to a much greater embedment than is needed. There are differences in opinion, of course, as to what constitutes the maximum energy per blow which will not harm a given pile. However, the writer has found that they do not invalidate the rule, since it gives results that are always on the safe side.

Application of this rule to piles which are to be driven into a hard stratum such as compact sand or gravel, hardpan, or to rock, results in no hardship, since there is little difference in length of the piles needed, whether the final resistance is governed by the rule or by reliable load tests.

Application of this rule to friction piles driven into a thick stratum of clay results in excessive embedment. Therefore the writer resorts to unconfined compression tests on 2-in. samples which permit at least an estimate of the lower limit for the unit friction that may be assumed safely. From this step a maximum embedment is computed, and is used together with the maximum resistance to which a given type of pile should be driven, in the following form:

> "The piles shall be driven to an embedment of so many feet, or to a resistance of so many blows per foot, with a hammer fulfilling specified requirements, whichever occurs first.

The writer has used this approach successfully on a number of projects.

One may object that pile formulas are also based, at least to a certain extent, on empirical knowledge. However, there are important differences:

(*a*) Empirical rules may be readily adjusted to conform with local soil conditions. A formula is too much respected as the product of sound theoretical thinking which must not be tampered with.

(*b*) An empirical rule will encourage engineers to look for more reliable information, particularly load tests, whenever sizable projects are undertaken.

(*c*) Empirical rules are given for certain soils, or specific types of piles, whereas formulas encourage extrapolation to entirely different conditions for which empirical knowledge is not yet available. Since pile formulas are theoretically not sound, extrapolation for new conditions is dangerous.

Summarizing, the writer believes that as a substitute for load tests, and to replace pile formulas, one or more of the following methods are practicable to

determine the allowable load and the resistance or embedment to which piles should be driven:

(1) Empirical rules for known soils of a given locality, derived from load tests and other experience (for example, when dealing with clays, rules for the allowable friction per square foot of embedded area from which the required embedment may be computed);

(2) For friction piles, when no empirical rules are available, determination of a lower limit for the allowable friction per square foot of embedded area, from unconfined compression tests on representative soil samples;

(3) Empirical rules for the maximum allowable loads for various types of pile, as given in building codes; and

(4) Empirical rules for the maximum number of blows per foot, of given energy per blow, to which the various types of piles should be driven, in the absence of other information, to permit the loads allowed under method (3).

A Difficult Task.—The question of how to treat the chapter on pile formulas is indeed a difficult one, particularly in view of the desired standard expressed in the first paragraph of the Manual manuscript: "This manual * * * endeavors to enunciate sound principles which are based on established facts, and to avoid stating rules or giving formulas which might lead to its unintelligent use." Rigorous adherence to this desirable goal would eliminate all pile formulas, since they are certainly not based on "established facts"; nor can one say that one can recommend any formula and feel reasonably sure that it might not "lead to its unintelligent use."

In closing, the writer wishes to express his appreciation to Admiral Bakenhus, the Chairman of the Committee, who with skill and patience steered his ship while some members of his crew, including the writer, were violently rocking the boat and more than once threatened to sink it. Whatever worthwhile results eventually come from the work of this Committee, the profession will owe special thanks to Admiral Bakenhus for his courageous effort.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EVALUATION OF FLOOD LOSSES AND BENEFITS

Discussion

BY JOHN B. MCALEER, JUN. AM. SOC. C. E., AND JOHN B. DRISKO, ESQ.

JOHN B. MCALEER,[17] JUN. AM. SOC. C. E., AND JOHN B. DRISKO,[18] ESQ.[18a]—Some of the basic concepts of flood losses are described in this most valuable paper. The author discusses benefits derived from the prevention of direct and indirect flood losses, on the basis of his experience. The method outlined cannot be applied indiscriminately, however, to each and every problem of flood protection. The evaluation of some types of benefit will require other methods. The author mentions some of these (for example, "social losses" and "diminution of capital value of real estate"), but does not include them. In some cases, there are intangible losses that are not included in the category of indirect losses, and there are benefits that do not spring directly from the elimination of a recorded loss. Proper evaluation of flood losses can be realized only by thorough consideration of the entire problem in a river basin coupled with detailed investigations and studies. Superficial studies usually result in partial or inconclusive results.

The author defines wealth and value as related to flood losses, based upon the *Encyclopædia Britannica*.[2] He makes no practical application of these concepts, however, and under the heading, "Collection of Damage Data," overlooks the determination of the value of the property to be protected. A knowledge of property values is essential to a common-sense view of protection. Determination of property values is required for "evaluation by estimating depreciated property value," a method briefly touched upon by Mr. Foster as an alternative method. Although not suggested by the author, it should be

NOTE.—This paper by Edgar E. Foster, Assoc. M. Am. Soc. C. E., was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Messrs. E. L. Chandler, E. F. Chandler, and Charles B. Burdick; October, 1941, by Messrs. H. K. Barrows, Roger E. Amidon, Hyman J. Fine, and Otto F. Buzhardt; and December, 1941, by Malcolm Elliott, M. Am. Soc. C. E.

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[18a] Received by the Secretary January 16, 1942.

[2] "Wealth," *The Encyclopædia Britannica*, 14th Ed., Vol. 23, p. 448.

noted that this method ("evaluation by estimating depreciated property value") may also be used to supplement the "annual loss method."

The purpose of this discussion is to bring out the practical application of value and wealth as (1) an approach to a method of rational determination of benefits beyond the method of the author, and (2) a "yardstick" for comparison with the cost of the protective works. The method to be discussed is an application of the approach referred to by Mr. Foster as "capital loss method." The discussion will attempt to clear the basic principles of valuation and increases in property value that result from flood protection—increases that are often quite independent of flood losses. A superficial study of the matter leads one to believe that such benefits are completely evaluated under the annual direct and indirect losses and benefits. The discussion citing the benefits of the Connecticut River Flood Control Plan is an example of this very common error.

Proceeding with the discussion of a rational method of determining losses and benefits, flood losses may be classified in three basic types: "direct," "indirect," and "intangible."

"Direct losses" are the physical damage to property and goods; the usual indexes are the costs of repair, replacement, temporary shifting of goods, and cleanup. Direct losses are caused by each individual flood.

The "average annual direct losses" may be determined by: (*a*) Summing up the total losses over a period of years and dividing by the period of record in years; or, (*b*) determining the average ordinate of a curve showing the relation between direct loss and the frequency. Such a curve is obtained by combining the stage-loss relation and the stage-frequency relation.

"Indirect losses" are the value or cost of the service, or the use lost or made necessary as a result of flood conditions. Losses of business, extra costs of rerouted transport, and other similar losses, both in the flooded area and remote from it, are all indirect losses. Indirect losses are caused by each flood.

The "average annual indirect losses" may be estimated as a proportion of annual direct losses, the proportion being indicated by investigations made for outstanding floods.

"Intangible losses" are the third type of loss. These losses are not susceptible of direct evaluation although they are sometimes of commanding importance. The intangible losses should be given separate statement and consideration. They are partly reflected in changes of property value and may often be evaluated from these changes. The total depreciation of property values as a result of floods reflects not only the capitalized direct and indirect losses, but also the lessened utility and other adverse effects of an intangible nature which determine the use, the value, and the future of property. These intangible losses may be evaluated as real losses of property value, and they are equal to the total depreciation minus the capitalized direct and indirect losses. They may be called "depreciation losses," as distinct from "total depreciation," and classed under a separate category. They are equivalent to a portion of the intangible losses. Recoverable depreciation losses are more practicably evaluated under equivalent benefits—that is, as "increases of property value," because the extent of recovery will vary with time of protection,

general economic conditions, prosperity of the region, demand for such properties, and condition of repair and use.

Intangible losses—including life, health, social and economic security, mental and physical distress—are caused by an individual flood but are intensified by a succession of floods—that is, by the flood history of a river. The knowledge that a certain area is subject to flooding casts a blight upon that area; and this causes a decrease in value which is often far greater than represented by the direct and indirect losses. Areas flooded frequently never develop fully, unless they are protected; areas visited for the first time by a record flood may suffer enormous depreciation, especially if the area is flooded or threatened more than once within a brief span of years.

Losses of the third class, intangible losses, are not susceptible of evaluation on the basis of recurring stage, or frequency. These intangible losses are reflected in real losses in the value and utility of property. Protection from floods will restore the value of such property and increase its utility. The benefits to be derived from the restoration of such values may best be classed as increase of property value.

The benefits of flood protection may be classed, then, as (*a*) direct, (*b*) indirect, (*c*) increases of property value, and (*d*) intangible benefits. Benefits (*d*) cannot be measured in dollars but include such items as prevention of loss of life, promotion of general welfare, and social security of the population. The benefits to be derived from a flood-control program are estimated on the basis of recurring losses which will be prevented, and upon the potential increases of value that may result from rendering recognized flood zones free of danger or from restoring highly developed zones that have been severely damaged by one or more unprecedented floods.

Direct and indirect benefits (Benefits (*a*) and (*b*)) are represented by the reduction in recurring losses. Many experienced losses are non-recurring by reason of altered usage or abandonment, or because replacement is combined with a betterment which renders future loss impossible or less likely. Recurring losses become less with each successive major flood, since property gradually becomes adjusted to the greater flood experience. Also, as a zone becomes recognized as a flood-dangerous zone, new capital is no longer introduced and the area may become stunted.

Increases of property value (Benefit (*c*)) are based on the higher utility and value that will result from adequate flood protection, insuring freedom from the disturbing disruption of possible flooding and increasing the net annual value or income of the property. Since many increases of property value that result from flood protection simply reflect the capital value of the direct and indirect losses prevented, benefits in this third class must be limited to a separate increment in value. Where property reflects important intangible losses or where the utility and value of property have been forced recently to a lower level by record flood experiences, increases in property values result from restoration to full utility by flood protection. These are equivalent to the restoration of depreciation losses. Often, benefits will result also from the more productive use of idle or partly developed lands where development has been retarded by floods, depending on future demand for such properties. In evaluating the

annual benefits due to increases in property value, each area and class of property presents a problem for individual analysis. The capitalized value of average annual direct and local indirect losses that have been experienced should be deducted where overlapping occurs.

After the benefits have been determined they are compared with the costs of protection. If benefits exceed costs, the project may have merit, but if benefits are less, the project is not economically justified.

A few examples will show the necessity for considering the value of the real estate in relation to the total value of the property to be protected:

Case (*a*).—Consider a lumberyard which is subject to frequent flooding, and suppose the annual cost of protection to be less than the annual value of the losses sustained. If the cost of protection is high compared with the value of land and buildings involved, however, the business should be moved to high ground and flood protection for this property should not be considered.

Case (*b*).—Consider a dwelling that is frequently flooded, and for which annual costs of protection are found to be less than the value of the losses. If the capital cost of flood protection is a large percentage of the value of the house (say, 50%), then protection of the property is not economically sound. Similar reasoning will apply to larger areas also. In certain cases where it is found that the justifiable cost of protection exceeds the value of the property Congress provides in the Flood Control Act of 1938, for evacuation of an area as an alternate to protection.

Case (*c*).—Compare two flood properties—one, a farm that is frequently flooded, and the second a hotel building that is rarely flooded. Although annual direct and indirect losses of the hotel are equal to those of the farm, it is evident that, were the owners of the property to be assessed for the cost of local protective works, the hotel management would be willing to pay more for protection than the farmer, simply because it has a much larger investment or value at stake. In this connection it is of interest to remark that in most cases where the cost of protective works has been borne by the community protected, assessments against the individual properties have been based upon a percentage of valuation or upon an increase in value rather than upon the damage-frequency relation.

Suppose the benefits do exceed costs. Then, is the project economically sound? Although the government finances most flood protection the project should be considered from a strictly economic point of view. Consider a group of business men viewing an investment, with costs to be assessed from the owners of property protected or the owner of an industrial plant considering protection of the plant against floods:

The cost of protection must bear a reasonable relation to (1) the total value or capital value of the property to be protected and (2) the value of real estate or fixed equipment which may not be relocated or moved out of flood danger. This is elementary. If either of these relations is high the project should be abandoned.

Consider the example of the lumberyard described in Case (*a*). Suppose protection is estimated to cost $30,000, whereas capitalized annual benefits amount to $35,000. Protection would appear reasonable because the total

worth of the establishment amounts to, say, $200,000; but closer inspection reveals that land and buildings are worth only $33,000, and the remaining worth is represented by stock. It is unreasonable to spend $30,000 to protect a bad investment of $33,000. The business should be moved to a more favorable site.

It is evident that the economic limit of protection should be set at some fraction of the value of the property to be protected, possibly 25%, and some greater portion of the value of real estate. Expenditures for flood protection should be held far below these limits unless benefits greatly exceed the cost of protection.

Consideration should be given to alternates to protective works: (1) Evacuation of the flood area, (2) altered use and zoning for future construction, (3) flood warnings, and (4) insurance.

The method of determining benefits, including increases of value, as outlined in this discussion, has definite advantages. It combines the precision of estimating benefits by the "annual loss" or "summation method" with the completeness of the methods based on "capital loss" or "differential" method. It is possible to determine, rationally, the "intangible" benefits which bear no relation to the direct and indirect losses and sometimes overshadow them. The method is no less accurate than any estimate of the worth of an industrial plant. The method is far more accurate than a computation by the annual loss method.

"Intangible benefits" based upon "depreciation losses" and representing increases of property value minus capitalized direct and indirect losses will vary in each locality and watershed and are generally found limited to densely populated growing urban and industrial areas or adjacent areas where development has been seriously retarded by floods. In areas that are retrogressing, in which there are idle mills, and in which industry and activity are shrinking rather than growing, properties become greatly depreciated by economic conditions and floods, but there can be little recovery of value. Even direct and indirect benefits in these areas based upon past experience represent an exaggeration. In agricultural areas benefits based upon direct and indirect losses are usually found to equal or exceed the total increase in value so that intangible benefits are zero.

Since September, 1938, flood-control investigations have been made by the Providence (R. I.) Engineer District on five rivers in the Providence District: The Blackstone, Pawtuxet, Thames, Connecticut, and Housatonic.

The principal industrial development along the Blackstone and Pawtuxet rivers has been textile, and these industries have not prospered in late years. Property values in these valleys have undergone economic depreciation, and recent floods have caused no recoverable depreciation.

The situation in the Connecticut Valley is quite different. The valley as a whole has prospered and activity is expanding. The valley has suffered three major floods in recent years—the 1927 flood, greatest for some seventy years; the 1936 flood, the greatest of record; and the 1938 flood, the second greatest of record on the main river and the greatest of record on the down-river tributaries.

This rapid succession of three great floods, during an otherwise favorable economic period of the valley, has caused substantial depreciation, a major part of which will be restored (and in some cases already has been restored) by proper flood-control measures.

Benefits accruing to the approved Connecticut River comprehensive flood control project have been analyzed by the method outlined herein. Annual flood control benefits are as follows:

Direct	$1,170,800
Indirect	1,029,100
Increases of property value	1,940,200
Total	$4,140,100

Capitalized at 5%, this is about $83,000,000 excluding incidental benefits to existing water power, navigation, recreation and pollution betterment, and to National Defense value, and without allowing for inevitable general growth and development of the area in the next fifty years, the period used for amortization of the works in all economic studies. The cost of the approved project is approximately $71,000,000. The total value of the property protected is approximately $856,000,000, and the real estate totals $596,000,000. Hence, benefits exceed the cost by 17%, and the cost of the project amounts to only 8.3% of the value of the property protected.

The increases of property value occur almost exclusively in the highly developed urban and industrial areas of the lower Connecticut Valley, where complete protection is afforded by the combination of a well-distributed reservoir system and the local protective works—that is, Northampton, Holyoke, Chicopee, Springfield, and West Springfield, in Massachusetts, and Hartford, and East Hartford, Conn. In Hartford, annual flood losses are as follows:

Direct	$ 191,800
Indirect	186,000
Depreciation	1,192,000
Total	$ 1,569,800
Capitalized at 5%	$31,396,000

It will be noted that depreciation losses are large in relation to direct and indirect losses. This is partly a result of the high value of the flood area (nearly $400,000,000) and partly because a dike built before the initiation of the federal project furnished protection against a flood having a probable frequency of occurrence of once in about fifty years and thus eliminated 60% of the annual direct and indirect losses. Had these low dikes not existed, the annual direct and indirect losses would have been about $950,000, and the depreciation would have been correspondingly less. The dikes, which had been constructed by local agencies, were topped in 1927, 1936, and 1938 with disturbing effect on property values in the flood areas. The "intangible" losses as measured by real "depreciation losses" are large and form the major part of economic justification for the dikes and walls which are now under construction. Bearing in mind that the initial benefits of stage reduction are credited

to the reservoir system, the remaining benefits based upon the annual loss method would not justify raising the dike in spite of the fact that Hartford was flooded three times in eleven years, and that direct and indirect losses in this period amounted to $19,000,000. By estimating increases in value a more nearly correct measure of the benefits is obtained.

The foregoing study was based on data obtained prior to 1939. Since that time much of the local protection has been completed. Experience has shown that the estimate of benefits was extremely conservative. In one large area in Hartford, land in the flood plane increased 300% in value before the dike protecting it was completed. A large municipal airport which was relocated at a field 5 miles from the center of Hartford after the 1936 flood was returned to its location in the city as soon as dike protection was completed. Expansion of housing and industrial plants within the flood area has progressed rapidly.

Hartford is typical of the other cities of the lower valley, although on a larger scale. Existing dikes at Northampton, Holyoke, Chicopee, Springfield, and West Springfield which afforded partial protection prevented many of the direct and indirect losses, which would otherwise have been experienced frequently.

In these communities, local protection works which could be justified by the annual direct and indirect benefits would be only half-measures, and would neither provide valuable flood protection nor promote any confidence in the value of the works. A true summation of benefits, however, including increases of property value which reflect the "intangible" benefits based principally on "depreciation losses," gives a more nearly complete determination of the real benefits, and justifies the construction of complete protection for these areas. Complete protection, in turn, insures the realization of the manifold benefits.

It is on such a sound basis that local works in the Connecticut Valley have been constructed, and even brief experience shows that anticipated results will be exceeded.

In conclusion, the economics of flood protection requires a thorough consideration of the entire problem in each river basin. A detailed determination of the valuation of the property to be protected is one essential part of such an analysis. In some river basins a thorough consideration of values will show the necessity of supplementing and extending the method outlined by the author to include benefits resulting from increases of property value. With this addition, the suggestions offered by the writer have more general application to river basins of varying character, and make possible a complete and rational evaluation of benefits. The result is a more accurate comparison of benefits with the cost of flood protection.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ALLOCATION OF THE TENNESSEE VALLEY AUTHORITY PROJECTS

Discussion

BY MALCOLM ELLIOTT, M. AM. SOC. C. E.

MALCOLM ELLIOTT,[3] M. AM. SOC. C. E.[3a]—The author has made a convincing presentation of the desirability of using the alternative-justifiable-expenditure theory as a basis for the allocation of costs of multiple-purpose projects.

It is assumed that this theory of cost allocation is applicable to preliminary studies of a project, made for the purpose of determining whether a project or any feature of it is economically warranted, as well as to the allocation of the known total cost after the project is completed. As far as preliminary estimates of costs, budgeting appropriated funds, and final accounting of expended funds are concerned, it appears that the alternative-justifiable-expenditure theory forms as good a guide as any that can be devised for determining what proportion of the total estimated or expended funds should be allocated to the various purposes. However, there is another use that must be made of the cost allocations of the TVA projects in which it is questionable whether the alternative-justifiable-expenditure theory is properly applicable.

The author has quoted from the TVA Act (see heading "Comment"):

> "* * * for the purpose of accumulating data useful to the Congress in the formulation of legislative policy in matters relating to the generation, transmission, and distribution of electric energy * * * the Board shall keep complete accounts of its cost of generation, transmission, and distribution of electric energy."

This language, the debates in Congress, statements in committee hearings, and arguments in lawsuits involving the TVA leave little doubt that the intent of Congress was in part to create a "yardstick" by which rates charged by private power companies could be measured. It goes without saying that the "yardstick" itself should be fairly measured; otherwise the measurements made with it will be unfair either to the public or to the private companies.

NOTE.—This paper by Theodore B. Parker, M. Am. Soc. C. E., was published in December, 1941, *Proceedings*.

[3] Col., Corps of Engrs., U. S. Army; Div. Engr., Upper Mississippi Val. Div., St. Louis, Mo.

[3a] Received by the Secretary December 31, 1941.

An important element in the cost of production of power is the cost of financing, including interest, the sale of securities, and amortization to extinguish the investment at the end of the useful life of the project. Naturally, these costs will vary with the amount of the investment. The cost of a dam built for power alone must be carried entirely by power revenues, whereas the cost of a dam built for power, navigation, and flood control is divided among these three purposes. Hence, the capital expenditures for power are likely to be much less in the case of a government-built, multiple-purpose project than in the case of a privately built, single-purpose power plant. In the example used in Table 1, the first cost of a single-purpose power plant is $600,000, whereas a plant of equal capacity constructed as a part of a multiple-purpose project is but $469,000, or about 78% of a single-purpose plant.

The private owner, of course, would base his rates on a first cost of $600,000; but the government, if it employs the alternative-justifiable-expenditure theory advocated by the author, will measure the private company rates with a "yardstick" based on a first cost of $469,000, and therefore might conclude that as much of the private rates as depends on financing and interest charges should be reduced by 22%. The foregoing is merely an assumed numerical example; but, on examining the cost figures as computed for actual projects (Table 2), even greater disparities are found between the cost of single-purpose (alternative) power plants and power features included in multiple-purpose plants. For example, considering the three-plant system in Table 2, the alternative power cost is $79,230,000, whereas the final allocation of the multiple-purpose cost to power is but $49,360,179—roughly 62½% of the alternative cost.

Many people in the United States are favorable to production of power by the government on the theory that power is such a vital public necessity that its production should be at the lowest possible cost and not burdened with the earning of profits for private investors. If such theory were the established policy of the government, as determined by Congress, the government properly would take all possible advantage of its unique opportunity of combining power with public uses, such as navigation and flood control, in a way to reduce the first cost of power installations even though private companies were "run out of business." In the TVA Act, however, Congress has not adopted such a policy. The evident purpose of the Act is to make use of the power produced in connection with navigation and flood control works, to market the power in order to help defray the cost of such works, and to accumulate data useful to the Congress in the formulation of legislative policy relating to production and marketing of power.

It is the view of the writer that the allocation of costs based on the alternative-justifiable-expenditure theory, as described by the author, will fall short of furnishing Congress the data it will need for the formulation of policies unless it is clearly explained that the first costs of power installations thus computed are not for comparison with the necessary and reasonable costs that must be incurred by private companies for single-purpose power plants.

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DISCUSSIONS

DESIGN OF ST. GEORGES TIED ARCH SPAN

Discussion

BY R. W. ABBETT, M. AM. SOC. C. E.

R. W. ABBETT,[4] M. AM. SOC. C. E.[4a]—A clear and concise statement of the basic theory of the design of an unusual type of bridge superstructure is presented in this paper. The fact that the principles of mechanics that are involved are not new to structural engineers does not detract from the usefulness of the explanation of their unique application. In all probability, however, the author's greatest contribution lies in the original procedure that he has developed for the preliminary design. The dimensioning of a statically indeterminate structure to initiate a stress analysis is a critical matter as far as labor and time are concerned, and the systematic procedure now available will be most helpful to future designers of similar arches.

As far as the writer is aware, the St. Georges Arch represents the first of its type in the United States, but several such structures were built in Europe before the beginning of the present war. It has certain definite architectural advantages when combined with deep girder approach spans, such as were possible and economical in this instance. A great deal of consideration was given to the appearance of the bridge because of its dominating location on Delaware's main highway.

As to the economy of the design, a comprehensive study of the cost of the arch span was made in comparison with other possible layouts for the St. Georges crossing, but few data are available for other span lengths. For spans of the order of 540 ft, the cost was estimated to be about the same as that of a Warren-type simple truss of the same length, and when combined with the girder approach spans this layout produced the least over-all cost of any considered for the high-level design. A low-level crossing with a vertical-lift main span, similar to the Buzzards Bay Bridge over Cape Cod Canal promised to be the least expensive of all, but this design was rejected by the client because of the heavy traffic on DuPont Highway and because of the frequent openings required for ship passages through the canal. A through cantilever

NOTE.—This paper by J. M. Garrelts, Assoc. M. Am. Soc. C. E., was published in December, 1941, *Proceedings*.

[4] Asst. Prof., Civ. Eng., Columbia Univ., New York, N. Y.

[4a] Received by the Secretary January 7, 1942.

truss and a self-anchored suspension bridge were considered for the main span. The costs of these two types were about equal and considerably greater than that of the arch or simple truss, principally because of the economy of short, simple-girder, flanking spans in lieu of the relatively heavy and expensive anchor spans required for both the cantilever and the suspension type. A tunnel was investigated and found to cost more than all bridge types. This was still the case when a two-lane tunnel was compared with four-lane bridges.

The fact that all piers were built on dry land effected the economy of the design in that substructure costs were reduced and shorter approach spans were possible in accordance with the economic principle of balancing the cost of substructure against cost of superstructure.

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DISCUSSIONS

FUNDAMENTAL ASPECTS OF THE DEPRECIATION PROBLEM A SYMPOSIUM

Discussion

BY MESSRS. PAUL T. NORTON, JR., H. L. RIPLEY, AND CARROLL A. FARWELL

PAUL T. NORTON, JR.,[27] ESQ.[27a]—There is so much to discuss in these three papers that it is difficult to find a place to start. Professor Grant has discussed prevalent fallacies in depreciation thinking,[28] but formulas containing these fallacies are still appearing in new textbooks and handbooks, as well as in articles in engineering and trade periodicals. The writer agrees with Professor Grant that this confused reasoning is probably less common in industry than in the published formulas, but this fortunate condition may not continue indefinitely. The next generation of industrial managers will include a much larger percentage of graduates of schools of engineering and business administration, who will probably not be nearly so unwilling to use mathematical formulas as were their predecessors.

Mr. Walls poses the question: "* * * is there not a very fundamental difference between the problem of measuring depreciation and estimating future life at a given time?" The two sentences that follow this quotation would seem to indicate a belief that the depreciation existing at a given time can be measured with comparative accuracy and reliability, but that estimates of future life are much less reliable. One also gets the impression that Mr. Walls believes that measuring depreciation existing at a given time and estimating future life are two separate and distinct problems. Depreciation means so many different things to different persons that such statements are difficult to discuss with assurance, but the writer believes that for most practical purposes these two things are merely parts of the single problem of deter-

NOTE.—This Symposium was published in November, 1941, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: November, 1941, by Messrs. Edwin F. Wendt and L. T. Fleming, and Anson Marston; and December, 1941, by Messrs. William G. Atwood, George E. Goldthwaite, Nathan B. Jacobs, H. J. Flagg, Nelson Lee Smith, and C. Beverley Benson.

[27] Prof., Virginia Polytechnic Inst., Blacksburg, Va.

[27a] Received by the Secretary January 22, 1942.

[28] "Principles of Engineering Economy," by Eugene L. Grant, Ronald Press, New York, N. Y., 1938.

mining present value, and also that the problem ordinarily cannot be solved without using both parts.

Messrs. Crum and Winfrey have given many reasons why there should be a better handling of government finance in relation to public works, but engineers should not forget that there are real dangers in the use of the capital-budget method for handling expenditures for nonliquidating public works. Many persons, including (unfortunately) a number of engineers and accountants whose duties include a consideration of the depreciation problem, seem to be obsessed with the idea that, if an expense is capitalized on the books and then amortized through depreciation charges, the cost is somehow transferred into the future. What an opportunity to befuddle the public! If it were only understood by every one that an investment in depreciating assets is a fully prepaid operating expense, this fallacy would be immediately apparent to every one who wanted to have a true picture of what really happens when so-called "capital investments" are made. That this is not merely an idle apprehension of the writer was proved in 1934[29] when it was proposed in Congress that depreciation rates on tax returns be reduced temporarily. This assumption, it was claimed, would not injure any one because very little equipment would probably be replaced during the period when the reduced rates were to be in effect. The idea that depreciation is actually paid for either when depreciation charges are made on the books, or when the asset is replaced, is obviously wrong; but this idea is held by many persons who claim to know more about depreciation than do the congressmen who could not see through the fallacy in the proposal just mentioned.

In making economy studies for all kinds of public works, liquidating or nonliquidating, engineers and economists should try, of course, to determine which projects will yield the greatest returns—sometimes merely a nonfinancial social return. In comparing alternatives it is probably best to use a method that converts the initial cost into an equivalent uniform annual cost of capital recovery with a return; but this can be done just as easily, and perhaps more safely, under present cash accounting methods. It should be remembered that the so-called "depreciation" that is used when making economy studies is entirely different from the depreciation charges made later on the books when investments are capitalized. In the case of public projects like power plants, where it is expected that all costs will be returned through charges for the service, it would seem that the same accounting methods should be used as are used in similar privately owned utilities, but one should not forget that the true financial results depend on conditions as they really are, and not necessarily on what the books make them seem to be.

Professor Grant refers all too briefly to the effects of the current depreciation tax policy on private decisions regarding plant expansion and retirement. In nearly all cases these effects seem to be bad when judged from the viewpoint of society as a whole and particularly from the viewpoint of National Defense. (It may be noted in passing that local property taxes tend to have similar bad effects.) As long as corporation tax rates were comparatively low, the de-

[29] See "Depreciation Under the Revenue Act of 1934," by Maurice E. Peloubet, *Journal of Accountancy*, Vol. 58, September, 1934, pp. 169–197.

preciation tax policy, although of questionable soundness, was of no great consequence. Until recently it could be stated without fear of contradiction that the economic aspects of depreciation were much more important than the tax aspects. It is not so certain that this is true today. Under present tax laws and regulations, it is not at all easy to secure approval of adequate depreciation charges on corporation income tax returns. This is a serious matter because every dollar by which depreciation charges are inadequate means an additional dollar of unreal but taxable profits, which are taxed in the highest bracket applicable to the particular company. That involves a rate as high as 72.4% under the Revenue Act of 1941, and every one expects that the rates will again be increased in 1942. In other words, many a corporation will be merely getting its investment back, but, because of inadequate depreciation charges, will be paying up to 72.4% of this recovered capital to the federal government. One would think that every top industrial executive in the United States would be trying to avoid this extreme penalty by making certain that his depreciation claims were adequate and were backed by such conclusive evidence that the U. S. Bureau of Internal Revenue would be certain to approve his claims. From having asked the question of many such executives, the writer believes that the majority do not even know that there is such a penalty.

From much that was printed and said when the special amortization provisions of the Second Revenue Act of 1940 were being considered in Congress, one would have thought that industry was asking for something for nothing. It would surprise many persons to learn that, even under these special provisions, industrial concerns in some cases may have to pay the 72.4% penalty on some of the money that is merely recovered investment and not really profit at all. In no case can a company do more than recover its investment, free of tax on such recovered capital; all real profits are fully taxable even under these special provisions.

Under present tax regulations a concern can scarcely hope both to get adequate depreciation allowances and to keep equipment in reserve for possible emergencies. Stand-by equipment is likely to be so costly in federal taxes that the average concern will hesitate to keep it for what may be possible but not very probable uses in the future. This particular tax penalty has probably not had much effect on equipment retirements to the present time (1942) because industrial executives are only beginning to realize that such a penalty exists; but the problem will be extremely serious in the future if the present tax policies are not modified.

H. L. RIPLEY,[30] M. AM. SOC. C. E.[30a]—One statement in the Symposium that is beyond dispute is the one appearing in the "Synopsis" of the paper by Professor Grant—namely, that "Writers on depreciation seem to agree on nothing except that other writers on the subject are somewhat confused." He might well have left out the "somewhat."

The writer agrees with the point raised by Professor Grant, to the effect that a clear definition should be had. This is easier to say than to do, and has

[30] Contract Agt., N. Y., N. H. & H. R. R., New Haven, Conn.

[30a] Received by the Secretary December 22, 1941.

not been done yet. He agrees also that depreciation estimates will enter in different ways into any proper solution of a concrete case, dependent upon the method of approach and the purpose and use to be made of the value after it is obtained.

It is being realized more and more that this word "depreciation," standing alone, has little or no fixed meaning that is recognized in common parlance, and the companion word "value" is in the same category; or, to state it another way, both words have many meanings dependent upon the concept, purpose, or intent of the user, and require a modifier to give them any clear standing in the mind of the reader. This last statement is borne out by the experience of the Interstate Commerce Commission, which found it necessary to make an interpretation of these two words when construing the language of the railroad valuation law, in which the bare words were used. In order to comply at all, the Commission coined such phrases as "valuation for rate-making purposes" (which phrase involved the matter of depreciation as a major factor) and "depreciation" (which was defined or described to fit the use made of it at the time it was introduced).

The position taken by the carriers' organization at that time (of which organization the writer was a member and so must share in the onus of any faulty presentation of principles or recommended practices) was confined also to a reiteration of a didactic assertion, and so perhaps was not very helpful.

Accounting practice is based upon the cost of a unit of property, expressed in dollars expended at the time of purchase or construction, without regard to the changing power in exchange of that dollar during the life in service of the unit of property which it represents, and the writer takes no issue with the propriety of such handling by accountants. When it comes to a question of finding a value at a specific date, however, it should be recognized that cost and value are not synonymous terms, but may be very different. When it comes to computing depreciation in obtaining present worth, that worth or power in exchange of the dollar is just as vital a factor as is the dollar itself. Both its power to purchase a replacement for the unit that has not yet reached the end of its useful life and the time that will elapse between the date selected for the appraisal and the day in the future when that unit will have reached the end of its useful life, and will, or should be, replaced, makes the date when the new dollar will have to be supplied a vital part of the problem. Because a dollar has earning power just as truly as does a machine or other piece of property that it represents, so the time at which the money must be provided as contrasted with the present date is a highly important factor, especially in connection with long-lived units, as present value is the objective of the inquiry. This is a truism that seems plain enough but is often ignored, as was done by the Interstate Commerce Commission. The Society recognized this fundamental principle in the Report of its Committee of 1917,[13] whereas the Commission handled it by the simple process of ignoring it in formulating its rules of procedure to direct its Bureau of Valuation.

All the foregoing means that there is a fundamental difference in the propriety of a method to be used in accountancy and of a method to be used in

[13] *Transactions*, Am. Soc. C. E., Vol. LXXXI (December), 1917, p. 1311.

finding the present worth in a going concern at a designated date for rate making, for purchase and sale, for condemnation of property, and perhaps for other purposes, as noted by other contributors to this Symposium.

Obsolescence and inadequacy frequently are more important factors in fixing the day when a piece of property is, or should be, taken out of service than wear or decay, although there are some exceptions such as railroad ties and the like. Obsolescence "happens"; it does not "accrue" in a straight line or any other line, and the same is true of "inadequacy" to a lesser extent. It is brought about by new inventions, suddenly, or by changed conditions, and so cannot be anticipated for any long period of future time with any close approach to precision. Faced with this stubborn fact, the appraiser, whatever method he may use, through study of past experiences, age-life expectancy tables, trained observation, and good judgment, or whatever tool or process he uses to guide him, must do the best he can and make the best estimate he can.

If he uses well-considered life tables as a tool to aid his judgment, he can proceed in an orderly manner, handling the problem step by step, and he can review his work, check his conclusions, and have some tangible basis for his final judgment. If he attempts to base his appraisal only upon general impressions gathered while viewing the great mass of unlike units that go to make up a railroad or a large industrial enterprise, no matter how experienced he may be, or how carefully he may study, or how sincere he may be, he is attempting that which is beyond the power of the finite mind. Any conclusions so reached will be little better than a "hunch," depending upon the experience and intelligence of the appraiser—or so it seems to the writer, who has tried it.

When a new appraisal of a later date is required, any circumstances that have developed to change the propriety of the previous forecast of life expectancy can, and should, then be studied and the forecast corrected. If the original appraisal is made by the step-by-step method, that can be done, but if it was made wholesale, by the general impression method, the writer doubts if a proper weight can be envisioned and applied to its effect upon the property as a whole in a way that would be consistent or convincing. Therefore, the writer agrees that age-life expectancy tables constitute a proper tool to be used in the solution of the problem.

If the step-by-step and age-life expectancy method is used in both appraisals, then the depreciation in the units as expressed in dollars can be transposed into its effect upon present worth by a simple application of proper factors. The Society's Special Committee on Railway Valuation has shown how to do it in its "Final Report." [31]

A table can be compiled readily at the rate of interest that money will earn at the time of the inquiry. Such a table will greatly facilitate the computations, if the computer will remember that the answer can be only an approximation at best. The answer is not mathematically precise from its very essence, because of the uncertainty inherent in the factors used in the solution by any process, and the fallibility of human judgment.

[31] *Transactions*, Am. Soc. C. E., Vol. LXXXI (December), 1917, p. 1464 *et seq.*, and Appendix I.

The important thing is to show the facts—all of them, and not just a part of them—and to handle them rationally in accordance with established principles that take into consideration all their essential implications and limitations. This must be done in a way that is susceptible of demonstration and analysis and can be checked for substantial accuracy. It should make no difference to the appraiser whether the property is publicly owned, privately owned, privately owned but under public control, or whatever its status, in computing the depreciation in physical property or the equivalent dollars that represent it.

The writer realizes that he has not solved the much discussed and controversial problem, and perhaps has not helped it, and that he has qualified many of his assertions because he is today much less prone to say "I know" than he was forty years ago, when he knew even less than he does now.

CARROLL A. FARWELL,[32] M. AM. SOC. C. E.[32a]—The writer wishes to reinforce Mr. Scharff's prefatory statement to the effect that it is important, early in the proposed program of study of the depreciation problem, to establish basic and simple definitions. The paper by Messrs. Crum and Winfrey is valuable in this particular, in addition to its emphasis on the desirability of the use of sound public accounting methods.

Why should there be given to the word "depreciation," when used without qualification, any meaning other than "decrease in value"? This has been called the "popular" or "dictionary" definition. Admittedly, under this definition, the terms "depreciation" and "value" are indeterminate unless qualified. Depreciation, or a rate of depreciation, may be estimated by an engineer or appraiser, or it may be established by the courts or other authority for a particular purpose. An accountant may make an allowance for depreciation on his books or in his financial statements; however, none of these actions determine depreciation but, instead, make an approximation to serve a particular purpose. The phrases, "estimated depreciation," "established depreciation," and "allowed depreciation," are not particularly cumbersome, and they tell the story without subterfuge. Such phrases as "ascertainment of depreciation," "correct depreciation," and "depreciation concept" will not be used if such an idea is followed.

Engineers sometimes hesitate to qualify their statements accurately as preciseness is often ridiculed, but the writer submits that "depreciation" (as well as "value") is indeterminate and, in using a definite figure to represent it, the engineer should make clear the relationship between the definite figure and the indeterminate quantity. If it is agreed that the unqualified word "depreciation" will be used only for the indeterminate quantity, the next step may well be an attempt to standardize methods of estimating, establishing, or allowing for, depreciation.

Most of the papers in the Symposium deal with the estimating or establishing of depreciation or depreciation rates and as such are valuable. However, it is important, first, that engineers agree as nearly as may be expected upon the definition of depreciation and go on from there to set up a correct and clear nomenclature.

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[32] Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

[32a] Received by the Secretary January 5, 1942.